Cyclic Loading Test for Circular Reinforced Concrete Columns Subjected to Near-fault Ground Motion

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**Abstract:** Near-fault earthquake load causes more serious damage in reinforced concrete (RC) structures when compared with far-field earthquake load. Particularly, a short column under the near-fault earthquake load is vulnerable to shear failure, which requires special design consideration to avoid the brittle failure under the near-fault ground motion. In the present study, quasi-static cyclic tests were conducted on four circular RC columns to investigate load-carrying capacity, deformation capacity, failure mode, stiffness degradation, energy dissipation capacity, and deformation components. For the test parameters, shear-span ratio, longitudinal bars diameter, and cyclic loading type were considered. The test results showed that under the cyclic loading to describe the near-fault ground motion, the RC columns were susceptible to premature concrete spalling and shear failure.

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Cyclic Loading Test for Circular Reinforced Concrete Columns Subjected to Near-fault Ground Motion

Wei-Jian Yi¹, Yun Zhou²*, Hyeon-Jong Hwang³, Zhi-Jie Cheng⁴, Xiang Hu⁵

Abstract

Near-fault earthquake load causes more serious damage in reinforced concrete (RC) structures when compared with far-field earthquake load. Particularly, a short column under the near-fault earthquake load is vulnerable to shear failure, which requires special design consideration to avoid the brittle failure under the near-fault ground motion. In the present study, quasi-static cyclic tests were conducted on four circular RC columns to investigate load-carrying capacity, deformation capacity, failure mode, stiffness degradation, energy dissipation capacity, and deformation components. For the test parameters, shear-span ratio, longitudinal bars diameter, and cyclic loading type were considered. The test results showed that under the cyclic loading to describe the near-fault ground motion, the RC columns were susceptible to premature concrete spalling and shear failure.

Key Words: Reinforced concrete column; Near-fault ground motion; Shear failure; Seismic behavior; Quasi-static cyclic test
Introduction

Near-fault earthquake loads such as 1994 Northridge, 1995 Kobe, and 1999 Chi-Chi earthquakes cause more serious damage in reinforced concrete (RC) structures than those of far-field earthquake load. Compared to the far-field ground motions that is considered in most of seismic design criteria, the near-fault ground motions show the distinctive response. Generally, the near-fault ground motions include a long period-high velocity pulse in the fault-normal direction and permanent ground displacement (Somerville, 2002). The analytical simulation for typical steel moment frames shows that the near-fault ground motions cause large displacement demands at the arrival of the velocity pulse, which require the structure to dissipate considerable input energy in a single or relatively few plastic cycles. The large displacement demand induces significant damage in the structures with limited ductility capacity (Kalkan and Kunnath, 2006).

Existing studies have mainly focused on the flexural behavior of RC columns under earthquake load. Park et al. (1985) proposed a model to evaluate the structural damage in RC structures under earthquake load. In the model, the structural damage was defined as a linear function of the maximum deformation and cyclic loading effect. Ang and Priestley (1989) tested twenty five circular columns subjected to axial and cyclic lateral loadings. It is reported that existing design equations cannot predict well the flexural capacity, and the initial shear strength of the columns were evaluated conservatively. When the displacement ductility exceeded 2.0, the shear strength was deteriorated with the increase of the flexural ductility of the columns. Watson and Park (1994) applied reversible quasi-state lateral loads to eleven RC columns to evaluate the analytical approach of the allowable flexural ductility, and the confinement effect on the flexural strength. Xiao et al. (1998) evaluated the shear strength of six high-strength concrete (HSC) columns subjected to cyclic load. They reported that the deformation capacity of the columns designed by the current design codes is governed by the flexural failure including concrete crushing and longitudinal bar buckling within the plastic hinge, instead of shear failure. Furthermore,
ASCE/ACI 426 recommendation (1978) and ACI 318 (2014) overestimated about 10% of the shear strength of the HSC columns showing shear failure. El-Bahy and Kunnath (1999) tested twelve identical quarter-scale circular RC bridge columns to investigate the relationship between the load path and cumulative seismic damage of the columns. Kazuhio and Park (2004) tested five RC bridge columns to evaluate the effect of loading pattern on the damage of the columns. Test results showed that the loading pattern was not critical to the maximum lateral load-carrying capacity of the columns, but the failure mode was affected by the loading pattern. Using the fatigue based damage model combined with energy dissipation, a simple procedure was proposed to predict the damage and failure of the RC columns subjected to an arbitrary seismic loading pattern. Phan and Saiid (2007) performed a shaking table test to evaluate the response of two large-scale RC bridge columns under impulsive near-fault ground motion and the residual displacements. They reported that near-fault earthquake records with forward directivity provide an asymmetric velocity pulse with high amplitude, which causes a whip-like behavior in the columns and large displacements in one direction. Phan et al. (2007) tested two flexural-governed RC columns under near-fault ground motion, and compared the test results with a similar column under far-field ground motion. The test results showed that the columns subjected to the near-fault loading exhibited the large asymmetric hysteretic responses. Choi et al. (2010) tested four cantilever-typed bridge columns under near-fault ground motion to evaluate the effect of near-fault earthquake load. It was concluded that the high amplitude velocity pulse associated with the near-fault loading caused large residual displacements of the columns. Liu et al. (2011) discussed the shear behavior of the short columns showing shear failure in the frame structure under near-fault earthquake load. Brown and Saiid (2011) investigated the effect of near-fault ground motion on substandard bridge columns and piers. Several large scale RC specimens were tested on a shake table using near-fault and far-field ground motion records. They reported that the near-fault ground motion caused larger strains, curvatures, and
drift ratios of the test specimens. The envelope curve of the test specimens subjected to near-fault ground motion was similar to those subjected to far-field ground motion.

In the last few decades, though shear failure of RC columns have often been reported in recent earthquakes, the shear behavior of the RC columns under earthquake load has not been a critical issue compared to the flexural strength and ductility of the RC columns. Particularly, as more near-fault ground motions have been recorded, careful considerations are needed for structures located in near-fault ground. As a fundamental study for RC columns subjected to near-fault ground motions, this study focused on the loading pattern effect on RC bridge columns that show shear failure. To investigate the effect of near-fault ground motions on RC bridge columns, four circular RC columns were tested under quasi-static cycle loadings on the basis of the near-fault and far-field ground motion records. The structural performance including load-carrying capacity, deformation capacity, failure mode, stiffness degradation, energy dissipation capacity, and deformation components were evaluated.

**Test Program**

**Test Specimens**

Fig. 1 and Table 1 show four RC column specimens in detail. The column specimens were designed according to Chinese codes and provisions (Chinese Code JGJ101-96, JGJ3-2010). The cross-section was circular shape with 370 mm diameter. The specimens were subjected to uniform axial compression and cyclic lateral loading. The test parameters were the shear span ratios (i.e., 2.56 for C-1 or 2.16 for C-2, C-3, and C-4), longitudinal reinforcing bars (i.e., 20-Φ16 for C-1, C-2, and C-3 or 12-Φ20 for C-4). The cyclic lateral loading procedure was to investigate the shear behavior under two-typed cyclic loadings (far field ground motion for C-1 and C-2 or near fault ground motion for C-3 and C-4).

For specimen C-1, column height was 1900 mm, which shows the shear span ratio 2.56. On the basis of the design method of the existing three test results showing flexural failure by Liu (2010) and Yi et al.
(2015), C-1 was designed to investigate the failure mode that changes from flexural failure to shear failure. Twenty HRB 400-D16 (GB50010-2010) bars (diameter Φ = 16 mm, cross-sectional area = 201.1 mm$^2$, and yield strength = 403 MPa) were used for longitudinal reinforcing bars. The rebar ratio was 3.74 %. HRB 235-D8 bars (diameter Φ = 8 mm, cross-sectional area = 50.3 mm$^2$, and yield strength = 397 MPa) were used for hoops. The vertical spacing was $s = 100$ mm, in which the hoops ratio was 0.5 %.

Specimens C-2 and C-3 had the same cross section and reinforcing bars as those of C-1. However, column height was decreased to 1600 mm, which shows the shear span ratio 2.16, to describe the short columns prescribed in Chinese provisions (Chinese Code GB50010—2010).

Specimen C-4 had the same cross section and column height as those of C-2 or C-3. However, twelve HRB 400-D20 bars (diameter Φ = 20 mm, cross-sectional area = 314.2 mm$^2$, and yield strength = 529 MPa) were used for longitudinal reinforcing bars. The rebar ratio was 3.51 %.

**Materials and Testing Setup**

At the day of the cyclic loading tests, concrete cube strength were $f_{c'} = 42.3$ MPa for C-1, 40.2 MPa for C-2, 51.1 MPa for C-3, and 39.9 MPa for C-4 (refer to Table 1). The yield and tensile strengths of rebars were $f_y = 403$ MPa and $f_u = 566$ MPa for Φ16 bars, $f_y = 529$ MPa and $f_u = 652$ MPa for Φ20 bars, and $f_y = 403$ MPa and $f_u = 566$ MPa for Φ8 bars, respectively.

Fig. 2 shows the cyclic lateral loading test setup. The top and bottom footings of the column specimens were post-tensioned to an L-shaped loading beam and reaction floor by tie-down rods, respectively. Two vertical actuators were connected to the L-shaped loading beam to apply constant axial load of $N = 1120$ to 1417 kN ($= 0.31$ to 0.33 $f_{c'} A_g$, in which $f_{c'}$ = cylinder compressive strength of concrete and approximately 80% of the cube compressive strength, $A_g$ = gross-sectional area of column) to the column specimens. Cyclic lateral load was applied using a horizontal actuator connected to the
mid-height of the L-shaped loading beam, which causes the inflection point at the mid-height of the
column specimens. The unbalanced moment caused by the eccentric loading of the L-shaped loading
beam was controlled by the two vertical actuators. At the beginning of each test, 20 % of cracking
loading was pre-loaded to verify the test facilities in proper working order. Deformations were measured
by linear potentiometers at the loading point and the plastic hinge zones. Strains of reinforcing bars were
measured by uniaxial strain gauges.

**Loading Plan**

Fig. 3 shows the two types of cyclic lateral loading applied to test specimens, which describe far-field
ground motion (type 1) and near-fault ground motion (type 2). The cyclic lateral loading protocols were
planned by the existing study by Orozco et al. (2002). Figs. 3(a) and (b) show the cyclic lateral loading
type 1 applied to specimens C-1 and C-2, respectively. Two load cycles were applied at every 0.25 %
lateral drift ratio increase. Fig. 3(c) shows the cyclic lateral loading type 2 applied to specimens C-3 and
C-4. Two load cycles were applied at initial 1 % lateral drift ratio, and then two load cycles were applied
at every 0.25 % lateral drift ratio decrease. After the load cycles reached zero, ten load cycles were re-
applied at 1 % lateral drift ratio. Finally, two load cycles were applied at every 0.1 % lateral drift ratio
increase.

**Test Results**

**Lateral Load-Drift Ratio Relationship**

Fig. 4 shows the lateral load-drift ratio \((P-\delta)\) relationships of the column specimens. The lateral drift
ratio was calculated by the lateral drift \(\Delta\) at the loading point over the column height \(H_c\) (= 1900 mm for
C-1, and 1600 mm for C-2 to C-4). Table 2 lists the peak strength \(P_u\), yield drift ratio \(\delta_y\), ultimate drift
ratio \(\delta_u\), and ductility ratio \(\mu\) of the specimens. According to Park (1988), the yield drift ratio \(\delta_y\) was
defined as \(P_u/(K_y H_c)\) (refer to Fig. 4(f)). The yield stiffness \(K_y\) was defined as the slope corresponding
to \(0.75P_u\). The ultimate drift ratio \(\delta_u\) was defined as the post-peak drift ratio corresponding to \(0.75P_u\). Fig. 5 shows the damage of the specimens at the end of the tests.

Figs. 4(a) and 5(a) show the test result of the specimen C-1. When the lateral force reached 187 kN at \(\delta = 0.5\%\), several flexural cracks occurred at the both ends of the specimen. The longitudinal bars yielded at \(\delta = 0.75\%\), and the yield load-carrying capacity \(P_y\) reached 235 kN and -262 kN. After the peak strength \(P_u\) reached 270 kN and -314 kN at \(\delta = 1.0\%\), hoop bars yielded, and diagonal cracks extended to the center of the specimen. Though spalling of the cover concrete occurred at \(\delta = 1.5\%\), the load-carrying capacity was not significantly decreased. At the first cycle of \(\delta = 2.0\%\), diagonal crack width increased to 3.8 mm. Ultimately, C-1 failed at the second cycle of \(\delta = 2.0\%\) because of concrete crushing at the top plastic hinge of the column.

In the specimen C-2, symmetric flexural cracks occurred at both ends of the specimen at \(\delta = 0.5\%\). At \(\delta = 0.75\%\), diagonal cracks occurred, and hoop bars yielded earlier than C-1. The longitudinal bars yielded at \(\delta = 1\%\), and the yield load-carrying capacity \(P_y\) reached 271 kN and -284 kN (see Fig. 4(b)). After the peak strength \(P_u = 312\) kN at \(\delta = 1.25\%\) and -321 kN at \(\delta = -1.0\%\), bond-slip cracks along the longitudinal bars occurred in the range of 500 mm at the bottom end, and spalling of cover concrete near the height of 50 mm was initiated (see Fig. 5(b)). Ultimately, C-2 failed at the second cycle of \(\delta = 1.5\%\) because of concrete crushing at the bottom plastic hinge of the column.

In the specimen C-3, flexural cracks occurred at both tension sides of the specimen at \(\delta = 0.3\%\). At \(\delta = 0.6\%\), the flexural cracks were extended to diagonal cracks, and new diagonal cracks occurred at 440 mm from the bottom end. At \(\delta = 0.8\%\), the diagonal cracks were occurred to the column center, and hoop bars yielded. The longitudinal bars yielded at \(\delta = 0.9\%\), and the yield load-carrying capacity \(P_y\) reached 357 kN and -435 kN (see Fig. 4(c)). After the peak strength \(P_u = 359\) kN and -441 kN at \(\delta = 1.0\%\), target drift ratio \(\delta\) was gradually reduced to \(\delta = 0.0\%\), and ten load cycles re-applied at \(\delta = 1.0\%\). In
the load cycles, the load carrying-capacity was almost maintained. However, spalling of cover concrete initiated, and hoop strain increased continuously. At the second cycle of $\delta = 1.1 \%$, the maximum crack width increased to 1.34 mm. Spalling of cover concrete occurred at $\delta = 1.4 \%$ (see Fig. 5(c)). Crack width in the plastic hinge exceeded 5.0 mm. The load-carrying capacity decreased to 134 kN and -168 kN in the positive and negative loadings, respectively. Ultimately, C-3 failed at the first cycle of $\delta = 1.5 \%$ because of concrete crushing at the plastic hinge of the column specimen.

In the specimen C-4, flexural cracks were initiated at $P = 180$ kN corresponding to $\delta = 0.2 \%$, and diagonal cracks occurred at $\delta = 0.5 \%$. The longitudinal bars and hoop bars yielded at 0.8 %, and the yield load-carrying capacity $P_y$ reached 299 kN and -341 kN (see Fig. 4(d)). After the peak strength $P_u = 302$ kN and -351 kN at $\delta = 1.0 \%$, load-carrying capacity decreased due to the ten times repeated cyclic loadings at $\delta = 1.0 \%$. At $\delta = 1.1 \%$, bond-slip cracks along the longitudinal bars occurred at the top and bottom end. At $\delta = 1.4 \%$, diagonal cracks expanded to the column center (see Fig. 5(d)). Ultimately, C-4 failed at the first cycle of $\delta = 1.5 \%$ due to shear failure.

In the specimen C-1, flexural failure is governed due to large shear span ratio 2.57. On the other hand, the specimens C-2 to C-4 with relatively lower shear span ratio 2.16 exhibited shear failure. The number of the flexural cracks was increased in the specimen C-1, while the number of the diagonal cracks was increased in the specimens C-2 to C-4.

**Deformation Capacity**

Table 2 lists the ultimate drift ratios $\delta_u$ and ductility ratio $\mu$ of column specimens. C-1 showing flexural failure due to large shear span ratio 2.57 exhibited the greatest deformation capacity $\delta = 2.0 \%$. On the other hand, C-3 showing shear failure due to lower shear span ratio 2.16 under cyclic loading type 2 exhibited the least deformation capacity ($\delta = 1.2 \%)$. This is because 1) low shear span ratio 2.16 increased the shear demand that increases the diagonal cracks and concrete damages in the specimen.
(see Fig. 5(c)); and 2) critical diagonal cracks were initiated at the first load cycle, which indicates the column specimen was susceptible to cyclic loading type 2 describing near fault ground motion. In C-2 under cyclic loading type 1 that describes far field ground motion, diagonal cracks were relatively well distributed compared to those of C-3. Even though the cyclic loading type 2 was applied to C-4, premature shear failure was prevented due to the reduced shear demand. As a result, the deformation capacities \( \delta = 1.5\% \) of C-2 and \( \delta = 1.5\% \) of C-4 were greater than that of C-3.

### Secant Stiffness

Fig. 6 shows the variation of the average secant stiffness at each load cycle. The secant stiffness \( K_s \) indicates the slope between the peak positive and negative strength in the load-drift relationship.

\[
K_s = \frac{|+P_i| + |-P_i|}{|+\Delta_i| + |-\Delta_i|} \tag{1}
\]

where \( P_i \) = peak strength at the \( i^{th} \) load cycle, \( \Delta_i \) = lateral drift corresponding to \( P_i \).

In Fig. 6, C-2 showed greater secant stiffness \( K_s \) than that of C-1 because of greater load-carrying capacity due to shorter shear-span ratio. The secant stiffness \( K_s \) of C-1 and C-2 decreased as the load cycles increased. This is because the existing flexural-shear cracks are expanded, and a few new cracks occur as the loading amplitude increases. The secant stiffness \( K_s \) of C-1 and C-2 at \( \delta = 1.5\% \) was approximately 66 \% or 63 \% of the yield stiffness \( K_y \). On the other hand, in C-3 and C-4, the secant stiffness \( K_s \) at the 1\text{st} cycle corresponding to \( \delta = 1.0\% \) was less than that of the undamaged secant stiffness \( K_s \) of C-2 at the 1\text{st} cycle. However, the secant stiffness \( K_s \) of C-3 and C-4 at the 1\text{st} cycle was almost same to that of C-2 at the 7\text{th} cycle corresponding to \( \delta = 1.0\% \). From the 3\text{rd} cycle to 8\text{th} cycle, the secant stiffness \( K_s \) increased because the lateral drift ratio decreased to \( \delta = 0.0\% \). After the ten times repeated load cycles at \( \delta = 1.0\% \), the secant stiffness of C-3 and C-4 showed 85.9 \% and 85.3 \% of those corresponding to the 1\text{st} load cycle, respectively. The secant stiffness \( K_s \) of C-3 at the ultimate drift
ratio $\delta_u = 1.2\%$ and C-4 at $\delta_u = 1.5\%$ was approximately 78\% and 83\% of the yield stiffness $K_y$, respectively.

**Energy Dissipation Capacity**

Fig. 7 shows the variation of energy dissipation capacity of the four specimens according to the number of cyclic loading. The energy dissipation coefficient $E_C$ per load cycle was defined as follows (JGJ101-96).

\[
E_C = \frac{E_D}{S_{OBE} + S_{ODF}}
\]  (2)

where $E_D$ = energy dissipation per load cycle defined as the area enclosed by a complete load cycle; $S_{OBE}$ = triangular area enclosed by points $O, B,$ and $E$; and $S_{ODF}$ = triangular area enclosed by points $O, D,$ and $F$.

C-1 and C-2 exhibited the similar $E_C$ values until the 10th load cycle. In C-2 showing shear failure, however, $E_C$ was measured until the 10th load cycle. On the other hand, after the 10th load cycle, $E_C$ of C-1 increased due to flexural behavior. C-3 and C-4 showed lower $E_C$ values than those of C-1 and C-2 because the cyclic loading initiated to the yield loading at the first loading step, which caused shear damage. After the 20th load cycle (i.e., $\delta = 1.1\%$), $E_C$ of C-3 increased due to strength degradation. On the other hand, $E_C$ of C-4 was not almost increased due to bar-slip. It should be noted that the large cyclic loading at the initial state decreased the energy dissipation capacity of the specimens.

**Strain of Hoops**

Hoops provide the confinement effect in the core concrete of a circular column, which increases the load-carrying capacity and deformation capacity. Furthermore, lateral buckling of the longitudinal bars are restrained by the hoops. Fig. 8 shows the strains of the hoops measured by strain gauges in the plastic hinge length. According to Priestley (1994), plastic hinge length $l_p$ of a flexural member can be defined as follows.
\[ l_p = 0.08l + 0.022d_b f_y \]  

(3)

where \( l = \) column length, which was considered as a half column height \( H_c \) because the both ends of the specimen were completely fixed; \( d_b = \) diameter of the longitudinal bars; and \( f_y = \) yield strength of the longitudinal bars.

In specimen C-1 showing flexural failure, hoop strains were concentrated at the height of 150 mm and 250 mm from the bottom end of the specimen, which was almost consistent with the calculated plastic hinge length 205.5 mm. At \( \delta = 2.0 \% \), concrete crushing at the bottom of the specimen caused the maximum hoop strain of about 0.0025 mm/mm, which was greater than the yield strain 0.0019 mm/mm.

In specimen C-2, after the hoops yielded at \( \delta = 1.0 \% \), the hoop strain significantly increased over 0.0140 mm/mm at \( \delta = 1.5 \% \). As a result, large diagonal cracks and concrete crushing failure occurred.

Specimens C-3 and C-4 exhibited similar hoop strain variation, except the maximum hoop strain locations at 250 mm and 150 mm from the bottom end of C-3 and C-4, respectively. After the hoops yielded at the first load cycle, crack width increased at the initial cracks. As a result, though the number of cracks of C-3 and C-4 was less than that of C-2, premature concrete damage and larger hoop strains were occurred (refer to Fig. 5).

**Evaluation of Structural Performance**

**Load-carrying Capacity**

The lateral load-carrying capacities of circular columns were evaluated by Chinese design code (GB50010—2010). Compression force \( N \) is applied to the column specimens, which decreases the lateral load-carrying capacity due to the second-order effect. Thus, the nominal strength \( P_n \) was calculated as follows:

\[ P_n = P_{no} - N\delta \]  

(4)
where $P_{no} = 2M_n / H_c$; $M_n$ is nominal flexural strength considering the compression force effect. $\delta = \text{lateral drift ratio of the specimens. The nominal flexural strength } M_n \text{ was calculated from section analysis considering the effect of compression force } N = 1210 \text{kN (C-1), 1180 kN (C-2), 1417 kN (C-3) and 1120 kN (C-4). The lateral load-carrying capacities } P_{no} \text{ of four specimens are 283kN, 330kN, 368kN, and 367kN, respectively. } P_{no} \text{ and } P_n \text{ of the specimens are presented as the dotted lines in Figs. 4(a)-(d). The lateral load-carrying capacities } P_n \text{ decreased as the lateral drift ratio } \delta \text{ increased due to the second-order effect. The predictions correlated well with the tests results.}

**Shear Strength**

The shear resistance of circular columns is provided by concrete, and hoops. According to Chinese design code (GB50010—2010), the nominal shear strength $V_n$ of the column specimen can be calculated with consideration of the contributions of concrete, transverse hoops, and applied axial load as follows:

$$V_n = \frac{1.75}{\lambda + 1} f_t b h_0 + f_{ys} \frac{A_{sv}}{s} h_0 + 0.07N$$

(5)

where $\lambda$ is shear span ratio; $f_t$ is tensile strength of concrete ($= 0.359 f_{c}^{0.55}$); $b$ and $h_0$ are width and effective height of a rectangular section, respectively, which can be replaced by $1.76r$ and $1.6r$ in the circular section ($r$ is the radius of the column section); $f_{ys}$ is yield strength of the steel hoops; and $A_{sv}$ is cross-sectional area of the hoops.

In test specimens, $V_n$ in Eq. (5) were greater than the shear demand $V_u$ by the peak strength ($V_n / V_u = 316/314 = 1.01$ for C-1, $318/321 = 0.99$ for C-2, $360/441 = 0.82$ for C-3, and $317/351 = 0.90$ for C-4). Since the ratio is less than 1.0 for C-2, C-3 and C-4, shear failure easily occur in the test specimens.

**Contributions to Lateral Drift**
The lateral drift $\Delta$ of the specimen consists of bond-slip deformation $\Delta_{bs}$, shear deformation $\Delta_s$, and flexural deformation $\Delta_f$ of the longitudinal bar due to the rotation of the bottom column (refer to Fig. 9).

$$\Delta = \Delta_{bs} + \Delta_s + \Delta_f$$  (6)

Bond-slip deformation $\Delta_{bs}$ is developed by the bond-slip of the longitudinal bars at the column-footing joint, which causes the rotation of the column as a rigid body behavior. Fig. 10 shows the bond-slip measurement and curvature distribution in the column specimen. In the present study, the uniformly distributed curvature was assumed in the measurement region.

$$\phi_1 = \frac{(\Delta_{L1} - \Delta_{R1})}{L_1H_1} \quad \text{for region 1} \quad (7a)$$

$$\phi_2 = \frac{(\Delta_{L2} - \Delta_{R2})}{L_2H_2} \quad \text{for region 2} \quad (7b)$$

where $\phi_1$ and $\phi_2$ = curvatures at the regions 1 and 2, respectively; $\Delta_{L1}$ and $\Delta_{R1}$ = displacements measured on left and right sides at the region 1 of the specimen, respectively; $\Delta_{L2}$ and $\Delta_{R2}$ = displacements measured on left and right sides at the region 2 of the specimen, respectively; $L_1$ and $L_2$ = horizontal distance between the linear potentiometer at the regions 1 and 2, respectively; and $H_1$ and $H_2$ = vertical distance between the linear potentiometer at the regions 1 and 2, respectively (= 30 mm and 100 mm).

Curvature $\phi_1$ at the region 1 includes both the flexural curvature and bond-slip curvature. Considering the relative small length $H_1$ of the region 1, the bond-slip rotation $\theta$ at the bottom of the column specimen can be calculated from the curvatures $\phi_1$ and $\phi_2$.

$$\theta = H_1(\phi_1 - \phi_2)$$  (8)

The bond-slip rotation $\theta$ is simplified to be developed at the center of the curvature $\phi_1$. Thus, the average bond-slip deformation $\Delta_{bs}$ is as follows.
\[ \Delta_{bs} = \frac{\left( \theta_{\text{top}} + \theta_{\text{bot}} \right)}{2} \left( H_c - H_1 \right) \]  

(9)

where \( \theta_{\text{top}} \) and \( \theta_{\text{bot}} \) = bond-slip rotation at the top and bottom ends of the specimen in Eq. (8), respectively.

As shown in Fig. 10, the bond-slip deformation of C-1 increased with the increase of load cycles. After the yielding of the longitudinal bars (i.e., 6\text{th} load cycle), the bond-slip deformation \( \Delta_{bs} \) significantly increased under positive loading, and decreased under negative loading. In C-2, the bond-slip deformation was less than those of other specimens. In C-3 and C-4, the bond-slip deformation \( \Delta_{bs} \) of C-4 was greater than that of C-3. This is because the specimen C-4 with the larger diameter of the longitudinal bar and low strength of concrete is vulnerable to bond failure.

Fig. 11 shows the shear distortion \( \gamma \) according to the load cycles. The shear distortion was calculated from the test measurement in the plastic hinge region of the specimens.

\[ \gamma = \left( \Delta - \Delta' \right) \frac{\sqrt{a^2 + b^2}}{2ab} \]  

(10)

where \( \Delta \) and \( \Delta' \) = diagonal deformations measured by the two diagonal linear potentiometers; and \( a \) and \( b \) = horizontal and vertical distance, respectively, between the ends of the diagonal linear potentiometers (= 400 mm and 400 mm).

The specimen C1 exhibited the almost same shear distortion to that of C-2 regardless of shear span ratio. In C-3, after the initial load cycle at \( \delta = 1.0 \% \), the shear distortion linearly decreased due to the reduced lateral drift. In the constant ten cycles at \( \delta = 1.0 \% \), the shear distortion increased slightly, but the increment was only 2.3 \%. Ultimately, the shear distortion of C-3 under cyclic loading type 2 was greater than that of C-2 under cyclic loading type 1.
Fig. 12 shows the ratio of each contribution of bond-slip deformation, shear deformation, and flexural deformation to the overall lateral drift measured from the test specimens. The flexural deformation was calculated by $\Delta_f = \Delta - \Delta_{bs} - \Delta_s$. The shear deformation $\Delta_s$ was calculated from the shear distortion $\gamma$. In C-1, after the yielding of the longitudinal bars at the 6th load cycle, bond-slip contribution decreased, but the shear deformation contribution increased. The contribution of the flexural deformation ranged from 2% to 45%. In C-2 with low shear-span ratio, the contribution of the flexural deformation was greater than that of C-1 because the bond-slip decreased. In C-3 with low $V_n / V_u = 0.82$, the shear deformation contribution was significantly increased due to shear failure. The contribution of the bond-slip deformation in C-4 was greater than that of C-3, which decreased the flexural deformation contribution. This result indicates that in C-4, the contribution of the bond-slip deformation was significant because of large diameter of longitudinal bars.

**Summary and Conclusions**

Four RC circular column specimens with different height and reinforcements were tested under two different cyclic loadings to investigate the seismic structural performance of the RC columns. On the basis of test results, load-carrying capacity, deformation capacity, failure mode, and energy dissipation capacity of the column specimens were evaluated. The primary test results are summarized as follows.

1. The specimen C-1 with shear-span ratio 2.56 exhibited flexural failure, which showed deformation capacity of $\delta_u = 2.0\%$. On the other hand, the specimens C-2 to C-4 with shear-span ratio 2.16 exhibited shear failure, which decreased deformation capacity of $\delta_u = 1.2$ to 1.5%.

2. The lateral load-carrying capacity of the column specimens predicted by Chinese design code (GB50010—2010) correlated well with the test strengths.

3. The number of flexural cracks in C-2 to C-4 was less than that of C-1, while the amount of diagonal cracks in C-2 to C-4 was greater than that of C-1. The number of cracks in C-3 and C-4 under the
cyclic loading that describes near-fault ground motion decreased, but the damage at the initial cracks increased shear deformation and decreased energy dissipation. This result indicates that near-fault ground motion decreases the seismic structural performance of circular reinforced concrete columns.

(4) The maximum hoops strain occurred between 150 mm and 250 mm of columns height, which were close to the plastic hinge length predicted by the empirical equation. The cyclic loading that describes near-fault ground motion increased the hoops strain due to strain concentration at the premature concrete damage, which increased the shear deformation.

Acknowledgement

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References


4 ACI Committee 318 (2014), “Building Code Requirements for Reinforced Concrete (ACI 318-14) and Commentary (318R-14),” American Concrete Institute, Farmington Hills, Mich.


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### Table 1 Test parameters

<table>
<thead>
<tr>
<th>Dimensions (mm)</th>
<th>C-1</th>
<th>C-2</th>
<th>C-3</th>
<th>C-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height (mm)</td>
<td>1900 (2.56)</td>
<td>1600 (2.16)</td>
<td>1600 (2.16)</td>
<td>1600 (2.16)</td>
</tr>
<tr>
<td>Longitudinal bars (steel ratio, %)</td>
<td>20-Φ16 (3.74)</td>
<td>20-Φ16 (3.74)</td>
<td>20-Φ16 (3.74)</td>
<td>12-Φ20 (3.51)</td>
</tr>
<tr>
<td>$f_y/e_u$ (MPa, mm/mm)</td>
<td>403 / 0.0020</td>
<td>403 / 0.0020</td>
<td>403 / 0.0020</td>
<td>529 / 0.0026</td>
</tr>
<tr>
<td>$f_u$ (MPa)</td>
<td>566</td>
<td>566</td>
<td>566</td>
<td>652</td>
</tr>
<tr>
<td>Hoops</td>
<td>Φ8@100</td>
<td>Φ8@100</td>
<td>Φ8@100</td>
<td>Φ8@100</td>
</tr>
<tr>
<td>$f_y/e_u$ (MPa, mm/mm)</td>
<td>397 / 0.0019</td>
<td>397 / 0.0019</td>
<td>397 / 0.0019</td>
<td>397 / 0.0019</td>
</tr>
<tr>
<td>$f_u$ (MPa)</td>
<td>504</td>
<td>504</td>
<td>504</td>
<td>504</td>
</tr>
<tr>
<td>Cyclic load$^a$</td>
<td>Type 1</td>
<td>Type 1</td>
<td>Type 2</td>
<td>Type 2</td>
</tr>
<tr>
<td>$f_c'$ (MPa)$^b$</td>
<td>42.3</td>
<td>40.2</td>
<td>51.1</td>
<td>39.9</td>
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<tr>
<td>Axial load (kN)</td>
<td>1210</td>
<td>1180</td>
<td>1417</td>
<td>1120</td>
</tr>
<tr>
<td>Axial compression ratio</td>
<td>0.32</td>
<td>0.33</td>
<td>0.31</td>
<td>0.31</td>
</tr>
</tbody>
</table>

$f_y$: yield strength; $e_u$: yield strain; $f_u$: tensile strength; and $f_c'$: concrete strength.

$^a$Type 1 and type 2 describe far-field ground motion and near-fault ground motion, respectively.

$^b$Cube compressive strength of concrete. On the basis of Chinese design code of concrete structures, the axial compressive strength of the concrete cylinder is approximately 80% of the cube compressive strength.

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### Table 2 Summary of cyclic loading test results

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<tr>
<th>Specimens</th>
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<th>Maximum drift ratio $\delta_u$ (%)</th>
<th>Nominal strength $P_{no}$ (kN)</th>
<th>Yield stiffness $K_s$ (kN/mm)</th>
<th>Ductility $\mu$ (= $\delta_u$ / $\delta_y$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1</td>
<td>+270</td>
<td>-314</td>
<td>+0.73</td>
<td>-0.76</td>
<td>+1.93</td>
<td>-2.01</td>
</tr>
<tr>
<td>C-2</td>
<td>+312</td>
<td>-321</td>
<td>+0.69</td>
<td>-0.61</td>
<td>+1.50</td>
<td>-1.53</td>
</tr>
<tr>
<td>C-3</td>
<td>+359</td>
<td>-441</td>
<td>+0.65</td>
<td>-0.51</td>
<td>+1.22</td>
<td>-1.20</td>
</tr>
<tr>
<td>C-4</td>
<td>+302</td>
<td>-351</td>
<td>+0.77</td>
<td>-0.51</td>
<td>+1.50</td>
<td>-1.54</td>
</tr>
</tbody>
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