Experimental and Analytical Investigations of Seismic Performance of Cantilever Reinforced Concrete Columns Under Varying Transverse and Axial Loads

Hakim Bechtoula*1, Susumu Kono2 and Fumio Watanabe3

1 PhD candidate, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan
2 Associate Professor, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan
3 Professor, Dept. of Architecture and Architectural Engineering, Kyoto University, Japan

Abstract

To assess the parameters influencing the seismic performance of plastic hinge regions in Reinforced Concrete (RC) columns, eight large-scale and eight small-scale cantilevered RC columns were tested under various vertical and horizontal loading patterns. Three different axial loads were imposed on the columns: a constant moderate load, a constant high load, and a varying axial load. In addition to this, there were three lateral loading patterns: uni-directional, square, and circular. The high axial load and bi-directional loading had a significant influence on the envelope curves as well as on the damage progress. The observed damage to large-scale columns was much more severe than that observed in the small-scale columns. The equivalent viscous damping for specimens under varying axial load was between the damping of the specimens under moderate to high axial load. An increase in the number of cycles caused a rapid degradation of the envelope curve of the load-displacement history. However, no significant effect was observed on the maximum lateral load carrying capacity, peak load. Analytical FEM results such as load-displacement, moment-curvature and axial strain shortening-curvature relations, closely matched the experimental results.

Keywords: column; reinforced concrete; cyclic loading; finite element analysis; damage

1. Introduction

The most widely observed damage to RC frame structures during an earthquake is the collapse of columns at the first-story. This type of collapse starts with the crushing of concrete, with subsequent loss of cover and buckling of the longitudinal reinforcement, which leads to a loss of axial bearing capacity. Understanding the parameters that influence the damage progress for a column during an earthquake is a primary issue.

Most of the previous experimental tests on reinforced concrete columns have been conducted under a constant axial load and a lateral monotonic or cyclic displacement or force (Stevens et al. 1991, Kowalsky et al. 1999). Li (Li et al. 1988) studied biaxial loading effect on building columns. Kuramoto (Kuramoto et al. 1995) tested high-strength concrete under triaxial loading effect. While the loading paths in these studies are complex, the applied axial load was proportional to the horizontal force. Due to effects such as vertical ground motion during an earthquake, the axial load on a column may not be constant. It is quite possible that the hysteretic characteristics of the columns, which include stiffness, strength, ductility and energy dissipation, are influenced by the axial load history.

A few researchers have experimentally studied the effects of variable axial loads on reinforced concrete columns (Abrams et al. 1987, Benzoni et al. 1996). Recently, Asad (Asad et al. 2004) carried out a test on six large-scale reinforced concrete circular columns. Experimental evidence revealed the significant effects of the magnitude and loading pattern of axial force on the seismic behavior of columns. They suggest that analytical tools, such as plastic hinge models, need to be modified to account for the effects of varying axial load.

The objectives of the research described in this paper were to experimentally and analytically address the problems associated with the performance of reinforced concrete columns subjected to various loading patterns in both lateral and vertical directions. This paper addresses the lack of data on the behavior of reinforced concrete columns subjected to bi-axial lateral load patterns under varying axial load.

2. Material characteristics and test setup

Eight large-scale and eight small-scale cantilever columns with square cross-sections were tested at Kyoto University under quasi-static unidirectional and bi-directional displacement-controlled horizontal loads with various axial loads. All specimens were designed to fail in flexure in the plastic hinge zone, based on the Japanese design guidelines (Architectural Institute of
Japan 1999). These columns represent models of the first-story columns of a real 6-story RC frame building that are widely used in Japan. Three hydraulic actuators applied orthogonal horizontal displacements at the top of the cantilever, as shown in Fig.1. Specimen dimensions and test variables are listed in Table 1.

The three horizontal displacement patterns were: linear reversed cyclic, circular cyclic, and square cyclic as shown in Fig.2. Small-scale specimens were loaded with 2 cycles at each of the following drifts: ±0.25%, ±0.5%, ±1%, ±1.5%, ±2%, ±3% and ±4%. The large-scale specimens were loaded with 2 cycles at each of the following drifts: ±0.25%, ±0.5%, ±0.75%, ±1%, ±2%, ±3%, and ±4%. The one exception to this loading scheme is specimen L2NVC, which was loaded with 4 cycles instead of 2 at each drift followed by two cycles at a lower rotation angle after every 8 cycles. Thus, the loading pattern for specimen L2NVC was 4 cycles at ±0.25%, 4 at ±0.5%, 2 at ±0.25%, 4 at ±0.75%, 4 at ±1%, 2 at ±0.75%, 4 at ±2%, 4 at ±3%, 2 at ±2%, and 4 at ±4%. The magnitude of the axial force was varied linearly as a function of the applied moment. The loading for specimen L1NVA is given in Eq. (1) and for L2NVA in Eq. (2):

\[
\frac{N}{f'_c D^2} = 0.3 + 2.47 \left( \frac{M}{f'_c D^3} \right)
\]

\[
\frac{N}{f'_c D^2} = 0.3 + 2.47 \left( \frac{M_{NS} - M_{EW}}{f'_c D^3} \right)
\]

where: \(N\) is the axial load, \(M\) the bending moment, \(D\) the depth of the column and \(f'_c\) the compressive strength of concrete. The constants 0.3 and 2.47 represent the effect of the dead load and the slope of the axial load variation on the normalized M-N interaction curve, respectively.

Concrete compressive strength as well as the longitudinal and transverse steel reinforcement are shown in Table 1.

3. Experimental Results

3.1 Load-drift relationships

Since shear failure was precluded during design, all specimens showed ductile behavior. Minimal difference was observed between the first-cycle and second-cycle envelope curves for specimens under a unidirectional load as seen in Fig.3. (a) for specimen D1N6. However, as illustrated in Fig.3. (b), a large difference is observed in the specimens under bi-directional load, even with a moderate axial load, 0.3Df'_c. Fig.3. (c) shows the normalized horizontal load versus the drift relationship for column L2NVC, which was loaded with four cycles at each prescribed drift. Under high axial load corresponding to the negative drift, a large drop in the normalized horizontal load is observed from the first-cycle to the fourth-cycle envelope curve. However, under a low axial load corresponding to the positive drift, difference was minimal between the second, third and fourth-cycle envelope curves. In the negative drift of the NS direction, L2NVB and L2NVC showed at 3% drift 20% and 50% reduction of the maximum horizontal capacity, respectively.

Fig.4. shows a comparison between the normalized horizontal load-drift curves of the small-scale D1N6 and the large-scale L1N6B under high axial load, 0.6Df'_c. D1N6 showed nearly the same normalized peak load values in positive and negative drift. In contrast to this, the negative peak-value for the large-scale specimen was only 75% of the positive peak-value. This difference is due to the more severe damage undergone by the large-scale column as compared to the small-scale column. A more detailed discussion of the observations of the specimens during testing follows. Small-scale specimens showed rapid strength degradation after reaching the peak. However, large-scale specimens maintained their maximum strength, showing nearly flat plateaus after their maximum peaks. As can be seen from Fig.4., both small and large-scale columns showed good energy dissipation with large hysteresis loops.
Fig. 3. Normalized Load-Drift Relationships

(a) D1N6

(b) D2N3

Fig. 3. Normalized Load-Drift Relationships (Continue)

(c) L2NVC

Fig. 4. Size-Effect on the Load-Drift Relationship

Table 1. Material Characteristics and Test Variables

| No | Specimen designation | Column width D (mm) | Shear span L (mm) | Concrete strength f'c (MPa) | Longitudinal rebar (ratio) | Shear rebar (ratio) | Axial force (axial force level in f'cD)^1 | Slope in normalized moment-axial force relation | Lateral loading directions |
|----|----------------------|---------------------|-------------------|-----------------------------|---------------------------|-------------------|------------------------------------------|-----------------------------------------------|--------------------------|
| 1  | D1N3                 | 250                 | 625               | 37.6                        | 12-D13                    | 2.44%             | 481MPa                                   | Constant (0.3)               | Uni                      |
| 2  | D1N6                 | 250                 | 625               | 37.6                        | 12-D13                    | 2.44%             | 481MPa                                   | Constant (0.3)               | Uni                      |
| 3  | D2N3                 | 242                 | 26.8              | 25.6                        | 12-D13                    | 2.60%             | 467MPa                                   | Constant (0.3)               | Uni                      |
| 4  | D2N6                 | 242                 | 26.8              | 25.6                        | 12-D13                    | 2.60%             | 467MPa                                   | Constant (0.6)               | Uni                      |
| 5  | D1NVA                | 250                 | 625               | 37.6                        | 12-D13                    | 2.44%             | 481MPa                                   | Constant (0.3)               | Uni                      |
| 6  | D1NVB                | 242                 | 26.8              | 25.6                        | 12-D13                    | 2.60%             | 467MPa                                   | Constant (0.3)               | Uni                      |
| 7  | D2NVA                | 242                 | 26.8              | 25.6                        | 12-D13                    | 2.60%             | 467MPa                                   | Constant (0.3)               | Uni                      |
| 8  | D2NVB                | 242                 | 26.8              | 25.6                        | 12-D13                    | 2.60%             | 467MPa                                   | Constant (0.6)               | Uni                      |
| 9  | L1N60                | 600                 | 1200              | 39.2                        | 12-D25                    | 1.69%             | 388MPa                                   | Constant (0.6)               | Uni                      |
| 10 | L1NVA                | 600                 | 1200              | 39.2                        | 12-D25                    | 1.69%             | 388MPa                                   | Constant (0.6)               | Uni                      |
| 11 | L2NVA                | 600                 | 1200              | 39.2                        | 12-D25                    | 1.69%             | 388MPa                                   | Constant (0.6)               | Uni                      |
| 12 | L2N6B                | 560                 | 32.2              | 12-D25                      | 1.94%                     | 388MPa            | 524MPa                                   | Constant (0.6)               | Uni                      |
| 13 | L2N6B                | 560                 | 32.2              | 12-D25                      | 1.94%                     | 388MPa            | 524MPa                                   | Constant (0.6)               | Uni                      |
| 14 | L2N6B                | 560                 | 32.2              | 12-D25                      | 1.94%                     | 388MPa            | 524MPa                                   | Constant (0.6)               | Uni                      |
| 15 | L2NVC                | 560                 | 32.2              | 12-D25                      | 1.94%                     | 388MPa            | 524MPa                                   | Constant (0.6)               | Uni                      |
| 16 | L2NVC                | 560                 | 32.2              | 12-D25                      | 1.94%                     | 388MPa            | 524MPa                                   | Constant (0.6)               | Uni                      |
3.2 Axial strain-normalized curvature relationships

Axial deformations of the column were measured over a gauge length equal to the column depth. Fig.5.(a) shows the axial strain variation of column D1N6 under a unidirectional horizontal load. Columns under high axial load, with either unidirectional or bi-directional horizontal loads, showed shortening during the entirety of the test. However, D1N3 under unidirectional horizontal load and moderate axial load showed both shortening and elongation during every cycle (Fig.5. (b)). Column D2N3, under bi-directional horizontal load and moderate axial load, showed only shortening during testing as shown in Fig.5.(c). In summary, for the columns tested in this project, there is a transition zone from 0.3 to 0.6 normalized axial load where the columns transition from tensile and compressive strains to only compressive strains under lateral loading due to the large damage that occurs in concrete. Based on the test results and considering retrofit concerns, we suggest lowering the maximum normalized axial load below the current 0.6 value suggested by the Japanese design guidelines (Architectural Institute of Japan 1999).

3.3 Variation of the equivalent viscous damping

The equivalent viscous damping factor of the first loading cycles for each specified displacement was evaluated for all columns. The equivalent viscous damping \( h_{eq} \) was calculated using the following equation:

\[
\Delta \theta = \frac{1}{4\pi} \frac{\Delta W}{W_e}
\]

where \( \Delta W \) is the area enclosed by one cycle of the hysteresis loop, and \( W_e \) is the equivalent potential energy.

Fig.6. shows a 3D plot of axial strain vs. normalized E-W and N-S curvature along with the projection on the curvature plane for column D2NVB. Typically, columns under bi-directional horizontal load exhibited more severe damage on the side subjected to compression due to flexure. This side coincided with the side that the "center of the projected loops", shown in Fig.6., moved toward.

Fig.5. Axial Strain Variation (Continue)
Buckling of a longitudinal rebar was observed at the N-E corner at a height of 0.5D. However, the corresponding small-scale specimen, column D1NVA, did not show any buckling until the end of testing. Concrete spalled around the column base to a height varying from 0.1D to 1.0D from the column base.

In testing specimen L2NVA, the maximum axial load was applied while the specimen was in the N-E quadrant while the minimum axial load occurred while the column was in the S-W quadrant. Cover concrete on the south face completely spalled off over a height of 1.5D. On the north side, all the longitudinal reinforcing bars buckled at heights ranging from 0.3D to 0.5D. One of the longitudinal reinforcing bars on the south side fractured at a height of 0.3D. In the corresponding small-scale specimen, D2NVA, the E-W corner bar buckled at a height of 0.2D. Concrete spalled at the base of this column over a height from 0.2D to 0.88D on all sides. Only three of the twelve longitudinal reinforcement bars did not buckle for specimen L2NVB. The buckled bars showed a double-curvature “S” shape at heights from 0.1D to 0.6D. Concrete spalled over a height range of 0.2D to 1.5D. The concrete was severely damaged, especially at the corners.

As an example, Fig.8. shows the final damage state of the small-scale D1N6 and the large-scale L1N60 columns. In the case of column L1N6B, concrete cover spalled first followed by buckling of the corner longitudinal reinforcement. As the test progressed, concrete at the corners started crushing, and the load carrying capacity gradually reduced as damage progressed toward the column core. The progression of damage from the outer cover to the inner core is confirmed by the strains in the transverse reinforcement shown in Fig.9. This figure shows the strain distribution of the inner and outer transverse reinforcing bars at 1% and 3% rotation angles. The strain in the main outer shear hoops decreases as the rotation angle increases from 1% to 3%. At the same time, the strain in the auxiliary inner shear hoops increases. The decrease in strain in the external hoops as the rotation angle increases corresponds to the damage caused to the concrete cover. This is further corroborated by the pure compression tests of Nishiyama (Nishiyama 1993) carried on RC columns. Nishiyama’s specimens had a 250x250 mm cross-section and high strength shear reinforcement. It was observed that the internal hoops yielded near the peak load while the external hoops remained in the elastic range.

Finally, as discussed above it can be concluded that the loading condition is a very important factor from the damage progression point of view as well as for the yielding sequence of the shear reinforcements.
developed at the University of Toronto in 1990. Since then, many experimental tests have corroborated the ability of VecTor2 to predict the load-deformation response of a variety of reinforced concrete structures exhibiting well-distributed cracking subjected to short-term static monotonic, cyclic and reverse cyclic loading.

VecTor2 is based on the Modified Compression Field Theory (Vecchio and Collins 1986) and the Disturbed Stress Field Model (Vecchio 2000) – analytical models useful for predicting the response of reinforced concrete elements subject to in-plane normal and shear stresses. VecTor2 models cracked concrete as an orthotropic material with smeared, rotating cracks. The program utilizes an incremental load, iterative secant stiffness algorithm to produce an efficient and robust nonlinear solution.

5.1 Finite Element Model

Even though a 3D model is appropriate for such analysis, however, due to the limitation of the VecTor2 program to 2D problems, the FEM analysis is limited to the specimens under unidirectional horizontal loading. Concrete was modeled using the four-node rectangular element as illustrated in Fig.10. The concrete cover was considered as plain concrete. To avoid convergence instability, the concrete around the applied axial load area was heavily confined. The reinforcing steel was modeled using a two-node truss-bar, and bond-slip was modeled using a four-node contact element. Table 2. summarizes the models used for the different parameters in this analysis. Details of the constitutive laws of the selected models can be found elsewhere (Toronto University 2002).

5. Analytical results

Two unique features of this project are the varying axial load and the bi-axial horizontal loading. To verify the varying axial load results, an FEM analysis was conducted to simulate the tests with varying axial load and uniaxial horizontal loading.

This analysis was carried out using the commercial nonlinear finite element program VecTor2 (Toronto University 2002), which was developed for two-dimensional analysis of reinforced concrete structures. The original version of this program, TRIX, was...
are compared in Fig. 13. for column D1N3; there is good correlation between the analytic and experimental results. Similar results were obtained for all other columns except column L1N6B, which had a large discrepancy between the experimental and analytical results. This is due principally to the fact that during the test and beyond a certain drift, the lower part of the column remained permanently deformed on one side whatever the loading direction was (Bechtoula 2002).

### 5.2 Load-displacement relationships

As an example, Fig. 11. shows a comparison between the experimental and analytical load-displacement relations for the large-scale column L1D60. In general, the correlation between the analytical and the experimental results is quite good. The results for the small-scale specimens are even better. Also, the analytical results for specimens under constant axial load matched the experimental results better than the analyses of specimens under varying axial load.

![Fig.11. Load-Displacement Relationship for L1D60](image)

### 5.3 Column top displacements and moment-curvature relationships

Fig. 12. shows a comparison between the applied column-top displacement and the computed column-top displacement for L1N6B, using the displacement gauges attached at the column base at a height equal to the column depth "D" (Park and Paulay 1975). The authors assumed that the flexural and shear deformations that contributed to the column-top displacement were those recorded at the column base for a height equal to the column depth.

In addition to column-tip displacements, analytical curvatures were evaluated over a distance equal to the column depth for all specimens. As an example, analytical and experimental moment-curvature curves are compared in Fig. 13. for column D1N3; there is good correlation between the analytic and experimental results. Similar results were obtained for all other columns except column L1N6B, which had a large discrepancy between the experimental and analytical results. This is due principally to the fact that during the test and beyond a certain drift, the lower part of the column remained permanently deformed on one side whatever the loading direction was (Bechtoula 2002).

![Fig.12. Comparison between the Applied and Computed Top Displacement for L1N6B](image)

![Fig.13. Moment-Curvature Relationship for D1N3](image)
5.4 Axial strain-curvature relationships

The FEM analysis was used to examine the axial strain vs. curvature relationship. The results for column D1N30 are presented in Fig.14. The axial strain was averaged over a gauge length equal to the column depth. The analytical results match the experimental results closely. The results shown in Fig.14, are typical except for specimens D1N60 and L1N60, which were both under high constant axial load. In these two cases, beyond 2% drift the analytical strains were nearly two-times the experimental values.

In addition to these FEM analysis results, other important analytical results using a fiber model related to these sixteen tested specimens can be found elsewhere (Kono et al., 2002).

4. Damage, concrete spalling and buckling, was more pronounced for large-scale columns than for small-scale columns. Shear reinforcement near the outer surface of the column yielded before the shear reinforcement closer to the core of the column. After severe damage to the external concrete took place, the strain in the inner hoops was higher than the strain in the outer hoops.

5. Analysis carried out using the Finite Element Program VecTor2, predicted with good accuracy the load-displacement, moment-curvature and axial strain-curvature for columns under a unidirectional horizontal load.

6. Conclusions

To assess the seismic performance of the plastic hinge region of reinforced concrete columns, sixteen small and large-scale models were tested under various axial and lateral loading patterns. The main conclusions of this experimental and analytical investigation are as follows:

1. Axial load intensity had a small effect on the envelope curve of the second-cycle of the load-displacement relation for specimens under a unidirectional horizontal load with a constant or variable axial load. However, specimens under bi-directional horizontal loads, showed a large difference between the first and the second envelope curves.

2. The type of loading, unidirectional or bi-directional, and axial load intensity had an influence on the axial strain variation.

3. Equivalent viscous damping increased with an increase in axial load. Columns under variable axial load, \((0 \sim 0.6)D^3f_c'\), showed an equivalent viscous damping value between those of columns under a constant moderate \((0.3D^3f_c')\), and constant high \((0.6D^3f_c')\) axial load.

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