13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 576

DYNAMIC RESPONSE CHARACTERISTIC OF REINFORCED CONCRETE COLUMN SUBJECTED TO BILATERAL EARTHQUAKE GROUND MOTIONS

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SUMMARY

This paper presents the dynamic response characteristics of a reinforcement concrete column subjected to bilateral earthquake ground motions. A Series of shaking table tests and fiber model analyses were conducted for 3 cross-sectional types of columns. The experimental results showed that the effect of bilateral excitation of the column was found to be significant on the non-linear response behavior of the column. Furthermore, it was found that the fiber element model analysis could simulate the experimental results well before the deterioration of the strength of the columns caused by the buckling of longitudinal reinforcement and the peeling of cover concrete.

INTRODUCTION

The structure behaves 3-dimensionally against earthquake ground motion, therefore the response characteristics is necessary to be reflected to the design of structures. As the first step of a breakthrough on the 3-dimentional Reinforced Concrete (RC) column response behaviors, we focus on the bilateral behavior. Several experimental and analytical studies on the RC column behavior subjected to bilateral excitation have been carried out. Hiraishi [1] showed that the nonlinear behavior of RC columns subjected to bilateral loading could simulate well using fiber element model analysis in comparison with the bilateral loading experiments. Takizawa [2] proposed the bilateral bending model using beam element model analysis based on the plastic flow rule, which incremental plastic strain vector increases to the normal direction of plastic potential function. Yoshimura [3] indicated that the fiber model and modified bilateral bending model analyses were correspond with the experimental results. These studies were based on the static loading tests results.

On the other hand, a few dynamic loading experimental tests including shaking table tests were carried out [4][5]. Therefore, analytical methods, which could simulate bilateral bending well for static case, are not verified sufficiently for the dynamic state like earthquake ground motions.

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This study presents the dynamic response characteristics of the RC column subjected to bilateral excitation based on the shaking table tests for the three columns and the fiber element model analyses.

SHAKING TABLE TEST SETUP

Test specimens and measurement condition

Three RC column specimens that had different cross-sectional geometry; square, circular and rectangular were designed. The scale factor was assumed as about 4 compared to the prototype structure. These specimens were designed with the same conditions as; longitudinal reinforcement ratio (square and rectangular: 0.95%, circular: 1.01%), height from the bottom of the column to the center of inertia (3,000mm), axial compressive stress at the bottom of the column (1.0 N/mm^2), hoop spacing (75mm), thickness of cover concrete (40mm) and design material properties. The longitudinal reinforcement ratio and axial compressive stress are determined based on typical highway bridge piers in urban areas in Japan. Steel weight was set up at the top of the RC column as auxiliary mass to apply for axial force according to law of scaling. This weight also produced horizontal inertia force. Nominal design strength of concrete was 27N/mm^2, and the nominal yield stress of bar was 295N/mm^2. Actual material properties are shown in Table-1. The diameters of bars ware 10mm for longitudinal bar and 6mm for hoop. The dimension of cross section was designed as 600×600mm for square, 600mm diameter for circular and 450×800mm for rectangular. The hoop was anchored by 135° hook for square and rectangular and by 90° hook with 240 mm lap length for circular. A cross tie was also installed in the rectangular column to make equal the theoretical confined effect of core concrete for each cross sectional principal axis. The column proportions are described in Figure-1.

Table-1 Actual Material Properties (Unit: N/mm^2)

Section Geometry	Concrete		Steel bar	
	σ_{ck}	E_c (×10^4)	σ_{sy}	E_s(×10^5)
Square	34.1	3.27	384	1.83
			350	1.85
Circular	33.7	3.30	372	1.79
			340	1.80
Rectangular	30.4	2.63	373	1.75
			316	1.64

Upper: longitudinal bar Lower: hoop

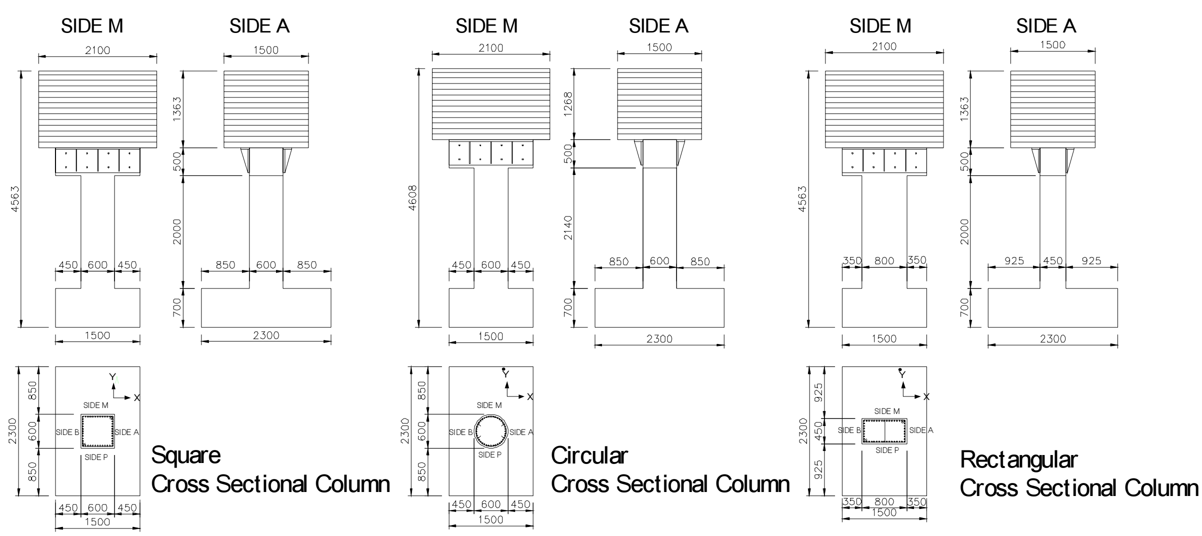


Figure-1 Column Properties

Response relative displacement (RD) and response acceleration (RA) of the 2 horizontal components were measured at the center of inertia. Displacement transducers were set up on the rigid steel flame on the shaking table. Moreover, accelerometers were set up at the same points to compensate for relative displacement between the rigid flame and shaking table. Accelerometers were also placed on the shaking table. The data sampling frequency was 200 Hz. The test setup was shown in Photo-1.

Test program

The JR Takatori Station record during the 1995 Hyogo-ken Nanbu Earthquake was used as the source of input waveform shown in Figure-2 [6]. Maximum ground acceleration was 642 gal for N-S component and 666 gal for E-W component. This waveform is characterized by inclusion of several large amplitude pulses.

The testing program consisted of two series for each column; small amplitude level (Run 1) and large amplitude level (Run 2). The amplitude levels were adjusted not to yield the longitudinal bar for Run 1 and to behave nonlinear response for Run 2 based on the fiber element model analytical results that were mentioned later. Time scale was reduced by a factor of 50% for all excitation cases. The amplitude scales were compressed to 20% and 100% (same as original waveform) for square, 15% and 80% for circular, 15% and 90% for rectangular for Run 1 and Run 2 respectively.

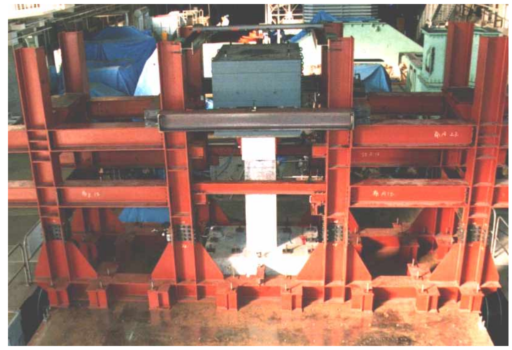


Photo-1 Test Setup

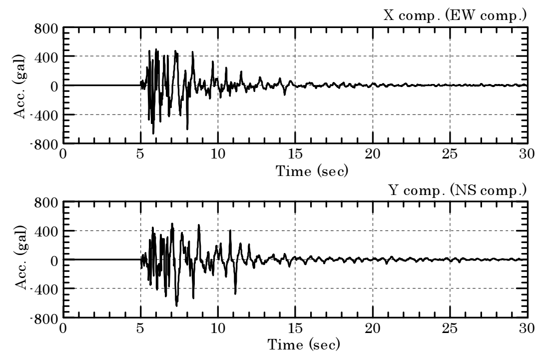


Figure-2 Input waveform (JR Takatori Station Record)

EXPERIMENTAL TEST RESULTS

Figure-3 and Figure-4 show the time histories and the orbits of the column at the center of inertia. The analytical results, which will be mentioned later, are also shown in these figures. Figure-5 shows the final damages of the columns.

At Run 1, no cover concrete was cracked and no longitudinal bar was yielded for all cases. During shaking in this stage, predominant frequency of the columns was stable. In square and circular column cases, shape of orbit of RA is similar to that of RD. In rectangular column case, RD for the weak stiffness axis of the structural cross section (Y axis) was obviously larger than that for the strong stiffness axis (X axis) though RA had not clearly predominant orientation.

At Run 2, the peeling of cover concrete and the buckling of longitudinal bars were developed while RD became maximum value (during 6 second to 7 second), though no bar fractured. In square column case, the damage area was concentrated up to about 500mm height above the footing. Horizontal cracks were developed in core concrete at 250mm height. All longitudinal bars were buckled though buckling length of the bars was varied with the location. The hooks of hoop anchored at 150mm and 225mm height, which corresponded with maximum curvature of longitudinal bar by buckling, were come off. Predominant frequency of the columns after severe damage was changed low in comparison with Run 1. The residual displacement was about 10mm for each cross sectional direction.

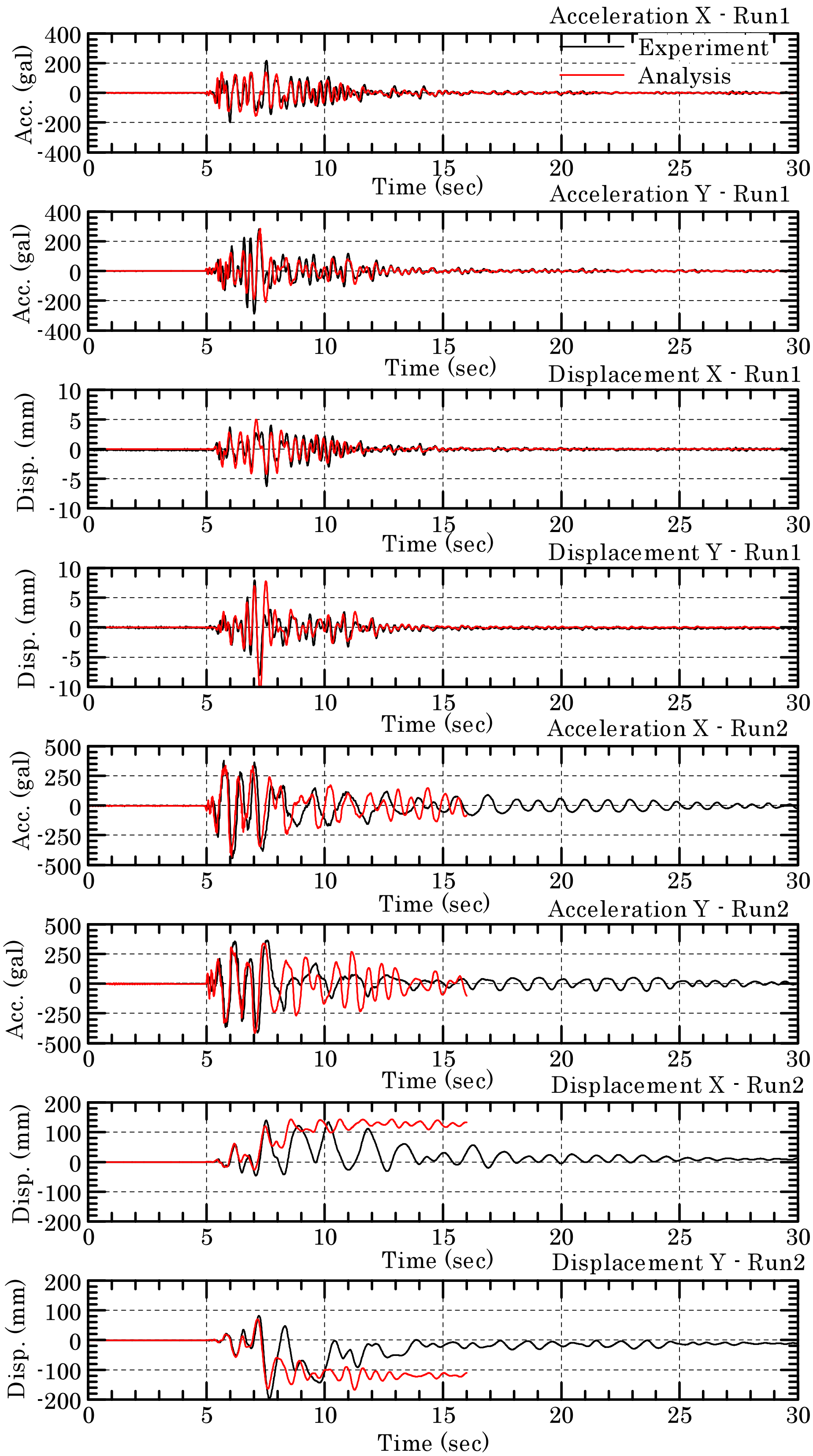


Figure-3 (1) Time histories of RA and RD at the center of inertia (Square cross section)

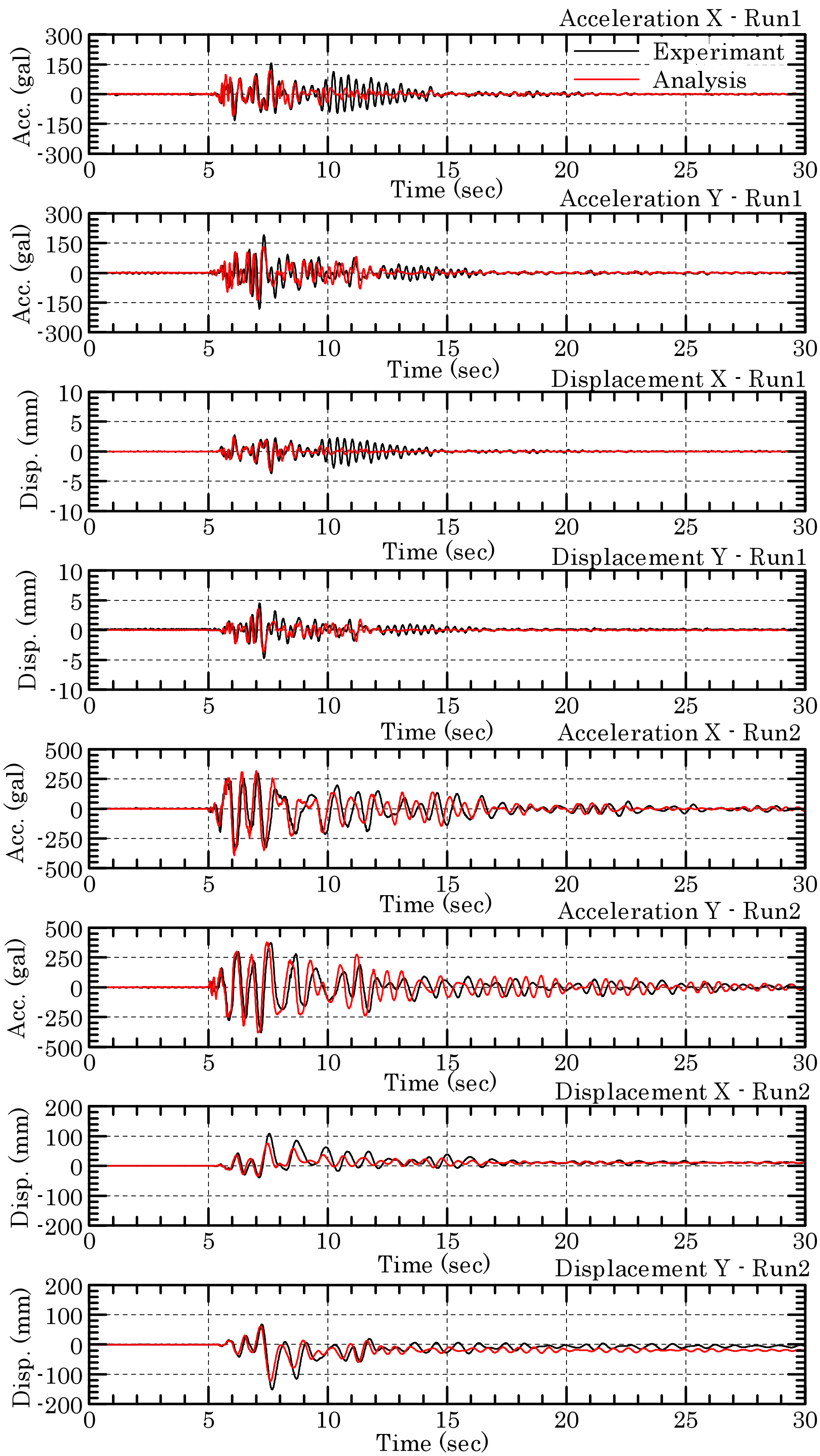


Figure-3 (2) Time histories of RA and RD at the center of inertia (Circular cross section)

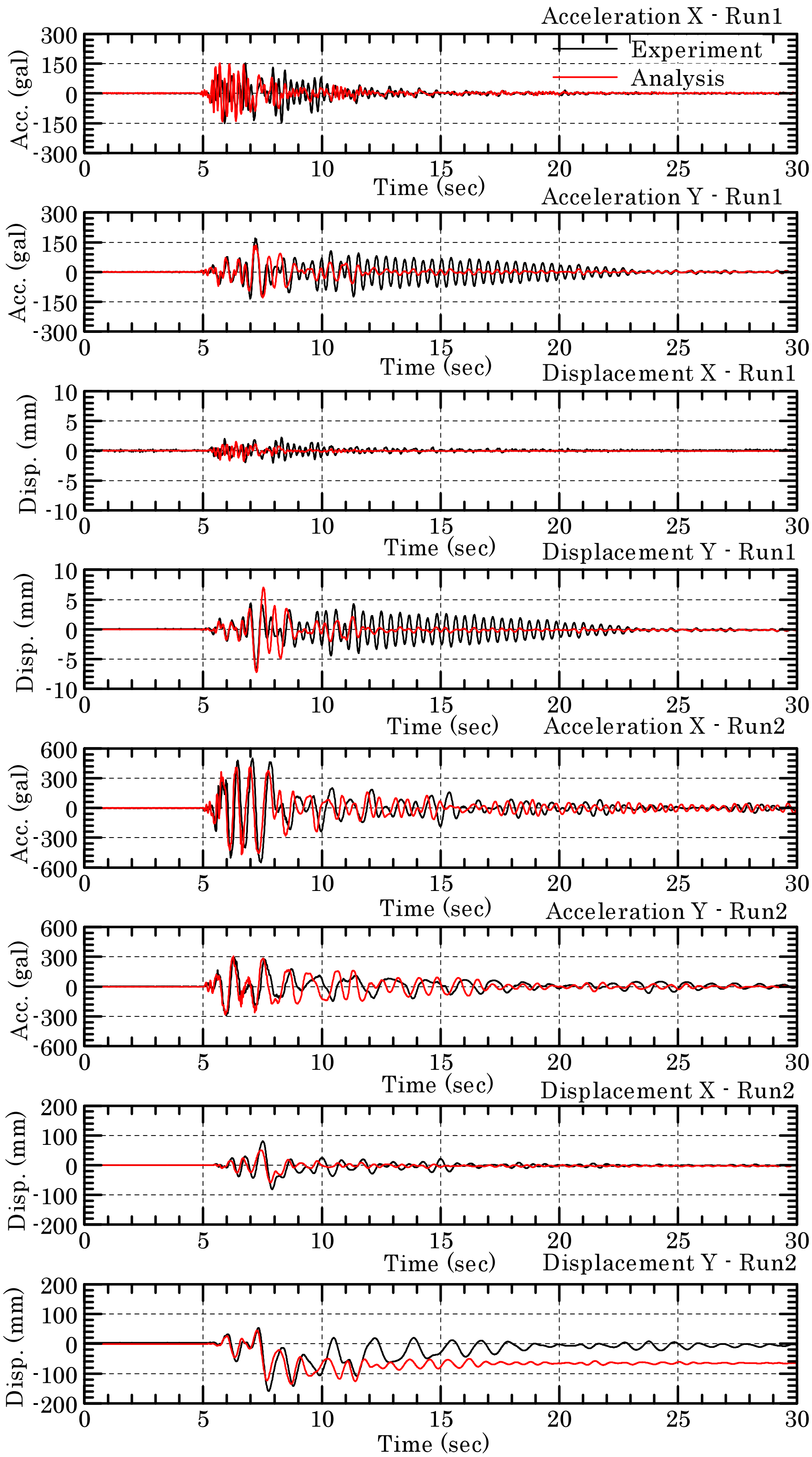


Figure-3 (3) Time histories of RA and RD at the center of inertia (Rectangular cross section)

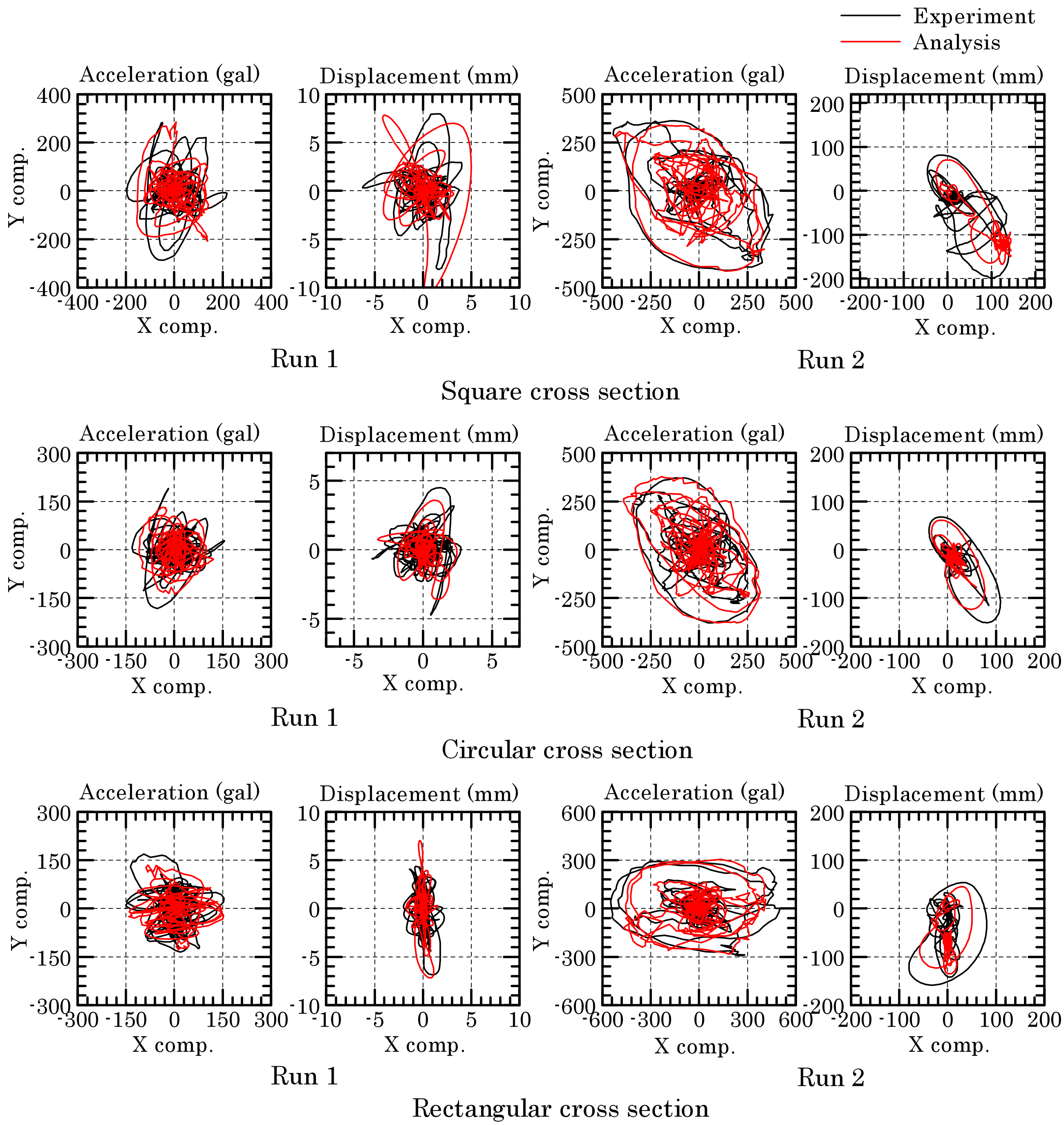
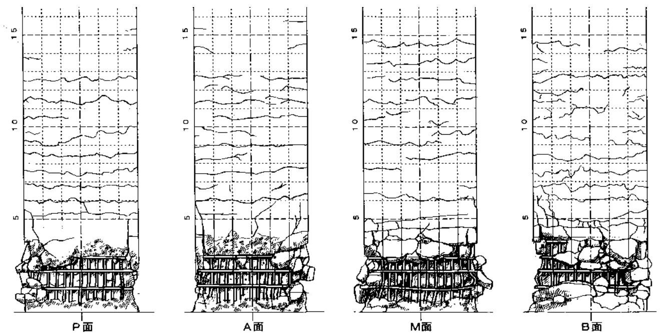
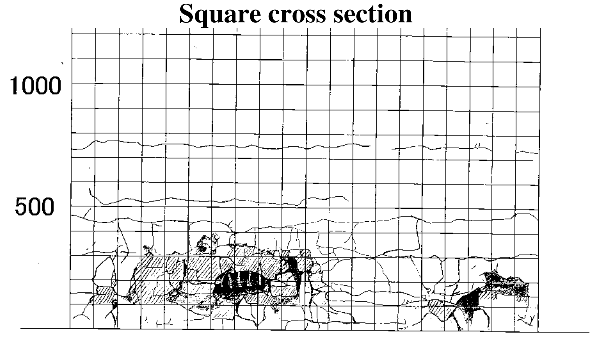


Figure-4 Orbits of RA and RD at the center of inertia

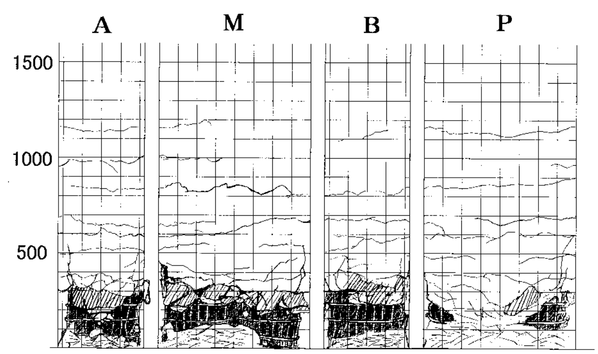
In circular column case, the damage was concentrated up to about 450mm height above the footing. Main horizontal cracks were developed at 300mm, 450mm and 750mm, though the core concrete had no damage. Cover concrete, which is located at the maximum RD direction side, was peeled in the area between 50mm and 350mm height. Fifteen longitudinal bars between 150mm and 225mm, which were equivalent to one leg of the hoop spacing, were buckled at the same area of peeling of cover concrete. The residual displacement was about 10mm in the X direction and 5mm in the Y direction.



Square cross section



Circular cross section



Rectangular cross section

Figure-5 Final damage of the columns

In rectangular column case, cover concrete was peeled up to about 400mm height. Horizontal cracks of cover concrete were developed at the 600mm and 850mm. The longitudinal bars were buckled except for 12 bars located at side P. The residual displacement was about 1mm in the X direction and 5mm in the Y direction.

The shapes of orbit of Run 2 were clearly different from those of Run 1. While the single large RD path was traced in Figure-3, RA path was circular or elliptic shape and the magnitude of vector sum of RA had approximately kept constant. This means that increment direction of RA was different from that of RD. This time was associated with severe damage that the peeling of cover concrete and the bucking of longitudinal bars were developed. Differing from Run 1, RA had predominant orientation in rectangular column case. Maximum RD for each principal axis of rectangular cross section at Run 2 was 81mm in the X direction and 159mm in the Y direction, therefore maximum RD ratio calculated for small response direction divided by large one was 2.0. At Run 1, this ratio was 3.5 so that RD for each principal axis was 2mm and 7mm. This ratio of Run 2 was decreased in comparison with that of Run 1, therefore the decreasing rate of stiffness of strong axis was larger than that of weak axis coinciding with deteriorating of the column. This means that the effect of the stiffness interaction for orthogonal axes is significant especially after deterioration of the column.

To verify the structural stiffness quantitatively, ambient vibration was measured. Structural predominant frequency, designated as *f*, could be estimated by comparing the Fourier spectral acceleration of shaking table to that of center of inertia. Substituting estimated predominant frequency shown in Table-2 into equation (1), the equivalent stiffness of the structure is obtained.

$$f = 2\pi\sqrt{\frac{W}{gK}} \qquad (1)$$

where, K= equivalent stiffness, W= weight of overall structure, g=gravity.

Table-2 Predominant Frequency of the Columns
(Unit: Hz)

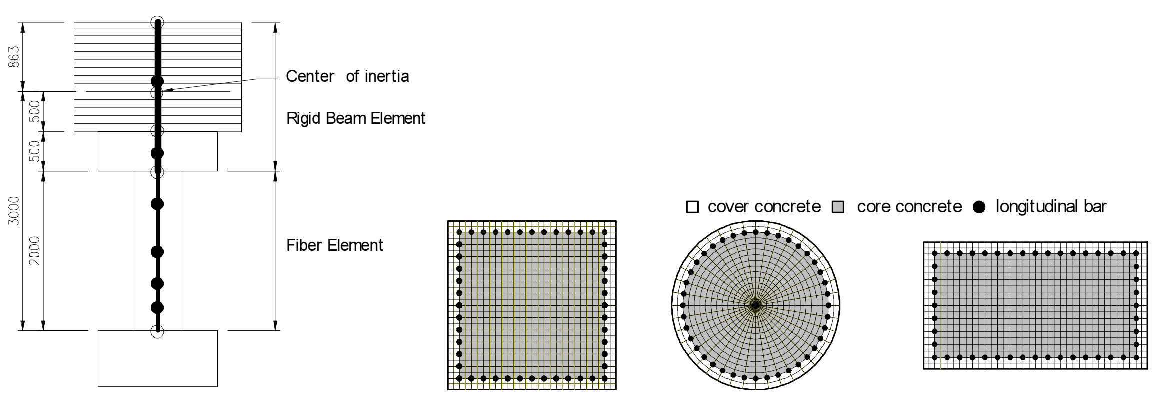
Section Geometry	Comp.	Before Test	After Run 1	After Run 2
Square	X	4.0	3.7	2.0
	Y	4.3	4.1	1.9
Circular	X	3.8	3.5	1.7
	Y	3.9	3.7	1.7
Rectangular	X	4.7	4.3	2.7
	Y	3.2	2.9	1.6

It was found that the column stiffness after Run 1 was approximately equal to initial stiffness. This result harmonized isochromatic response wave. The stiffness after Run 2 was deteriorated about 30% in comparison with initial stiffness.

SIMULATION ANALYSIS OF THE EXPERIMENTAL TEST

Analytical Model

Figure-6 shows the analytical models and fiber separation of cross section. Analytical model consists of fiber element for the column and rigid beam element for the beam and steel weight. The acceleration waves recorded on the shaking table are applied directly at the base of the column as input motion. The structural response is sensitive for fiber element arrangements of member axis especially in the hinge region. In this study, fiber element length at the base of the column was assumed as 300mm. This length was defined based on the plastic hinge length in Japanese highway seismic design specification [7].



Square **Circle** **Rectangle**

Column model **Fiber separation**

Figure-6 Analytical model

The same material properties for the analyses were used as actual ones listed in Table-1. The stress-strain relationships were assumed as Kent and Park model [8] for cover concrete and Hoshikuma model [9] for core concrete. The relationships after $0.2f_c$' were defined as shown in Figure-7. Tensile side was modeled as linear until tensile fracture strength. Ristec model [10] was used for the hysteresis of the concrete. The stress-strain relationship of longitudinal bar was assumed as modified Menegotto-Pinto (MP) model proposed by Sakai and Kawashima [11]. They pointed out that original MP model [12] had characterized to overestimate of the stress in case of reloading after small amplitude unloading, so they modify loading path in case that reloading after unloading is accordance with previous hysteresis curve as following conditions;

$$\sigma_r \geq 0 \text{ (strain increased case)} \quad (2)$$

$$\sigma_r \leq 0 \text{ (strain decreased case)} \quad (3)$$

where σ_r = stress at the reversing point.

The original and modified MP models were shown in Figure-8.

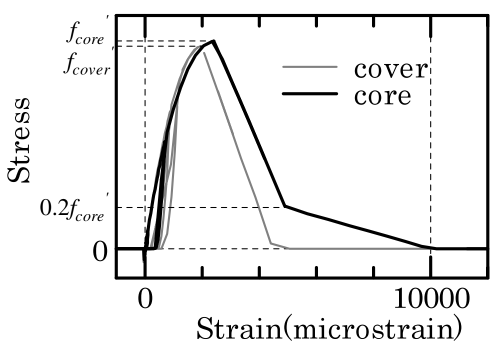


Figure-7 stress-strain relationship (concrete)

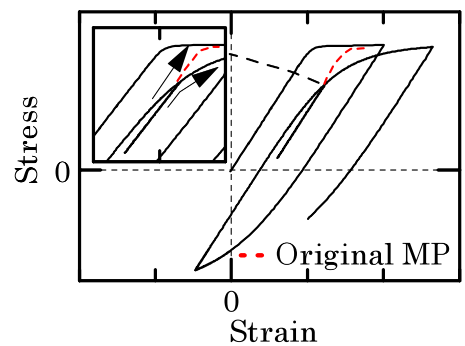


Figure-8 stress-strain relationship (steel bar)

Damping constant was assumed as 2% for all elements. Rayleigh damping was assumed as overall damping matrix. Table-3 shows the modal analysis results. Adjacent modal frequencies are slightly different at the square and circular columns because of anisotropic of steel weight. Therefore, these modes could be essentially regarded as equal, so the average of the 1st and 2nd vibration modes is defined as the 1st mode for the overall structure. Identically, the average of the 3rd and 4th vibration mode is defined as the 2nd mode for the overall structure. In rectangular column case, the 5th vibration mode is used instead of the 4th mode which is dominated by vertical vibration mode (Z comp.). These vibration modes for the overall structure were used to determine Rayleigh damping coefficient.

Table-3 Modal Analysis Results (1st - 5th mode)

Mode	Square		Circular		Rectangular	
	Freq.(Hz)	Comp.	Freq.(Hz)	Comp.	Freq.(Hz)	Comp.
1	5.08	T-Xcomp.	4.46	T-Xcomp.	3.52	T-Ycomp.
2	5.16	T-Ycomp.	4.53	T-Ycomp.	6.09	T-Xcomp.
3	54.8	R-Ycomp.	46.3	R-Ycomp.	45.9	R-Ycomp.
4	64.3	R-Xcomp.	57.0	R-Xcomp.	60.2	T-Zcomp.
5	66.6	T-Zcomp.	65.8	T-Zcomp.	61.6	R-Xcomp.

T: Translation R: Rotation

Time history response analyses were carried out using Newmark β method (β=0.25). Integration time step was 1/2000 second. Unbalanced force caused by the differences of stiffness at previous and current time step was carried over the next step. P-δ effect was considered to add geometric stiffness matrix in proportion as the initial stiffness.

Comparison between experimental and analytical results

The time history and the orbits of the experimental and the analytical RA and RD are shown in Figure-3 and Figure-4. At Run 1, the experimental and the analytical results are agreed well for all time. At Run 2, these are also agreed well until maximum RD is occurred. In case of the square column, the analytical results could be obtained until 16 second by reason of convergence condition and large residual displacement remained.

In the strict sense, the analytical RD was smaller than the experimental one for all cases especially in nonlinear response. On the contrary, the analytical RA was larger than the experimental one. Following these results, the stiffness of analytical model is greater than that of experimental one. The time that analytical time history diverged from the experimental one corresponded to be developed the damage such as the buckling of longitudinal bar and the peeling of cover concrete. The failure mechanism after this stage is different from the fiber element model's assumption that cross section of the member kept plane and materials between concrete and steel bar were bonded completely. The incremental direction of RD and RA were also simulated well, hence the maximum RD direction of the column subjected to the bilateral excitation could be estimated by the fiber element model analysis. This result indicated that the column behavior subjected to the bilateral excitation could not be simulated without considering the stiffness interaction between horizontal principal axes.

CONCLUSIONS

Following conclusions are derived from this study:

1) Shaking table tests were carried out for 3 RC columns. The experimental results showed that incremental vector direction of RA was different from that of RD and magnitude of vector sum of RA had approximately kept constant in nonlinear response.
2) Analytical method using fiber element model could be simulated the experimental dynamic response results well until damage of the column such as peeling of the concrete and buckling of the longitudinal bar was occurred.

ACKNOWLEDGEMENT

Special thanks to Dr. Jun-ichi Sakai, postdoctoral researcher of PEER, UC Berkley, contributes to us the source code of modified MP model. This source code is released his website.

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