HYSTERETIC BEHAVIOR AND DYNAMIC RESPONSE OF RE-CENTERING REINFORCED CONCRETE COLUMNS

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1. INTRODUCTION

In recent years, a high ductility capacity is expected of bridge columns located in regions of high seismicity, like California and Japan, to ensure economical designs with adequate protection against collapse during strong ground shaking. It has been noted, however, that bridge columns that develop high ductility demands during extreme ground shaking are likely to retain large residual displacements following the earthquake. To maximize post-event operability and minimize repair costs, attention should be paid in the design process to minimizing these residual displacements.

As a result of damage suffered by bridges in the 1995 Hyogo-ken Nanbu earthquake, the Japanese Design Specification of Highway Bridges was revised in 1996 and included a requirement for limiting the residual lateral displacement at the top of a column after an earthquake [1]. In an effort to satisfy the specification, some research has focused on developing new systems to reduce the residual displacements of reinforced concrete bridge columns [2),3)].

In the research described in this paper, a series of static cyclic analyses for reinforced concrete bridge columns was conducted to study how to mitigate column residual displacements, followed by a series of dynamic analyses to validate the effectiveness of this approach.

2. BRIDGE COLUMN ANALYZED

A reinforced concrete bridge column designed in accordance with the Caltrans SDC [4] was analyzed. **Figure 1** shows the dimension and the cross section of the column. The column was 1.83 m in diameter and the height from the bottom of the column to the gravity center of the superstructure was 10.97 m, resulting in an aspect ratio of 6. The dead load supported by the column P was 4.5 MN, and the unconfined concrete strength f'_{co} was 34.5 MPa. Thus, the ratio of axial load to axial load capacity $P/f'_{co}A_g$ (axial load ratio) was 5%.

The column was reinforced with 48 No. 9 (29 mm-diameter) deformed bars and No. 5 (16 mm-diameter) spirals at 76 mm pitch. The longitudinal reinforcement ratio, ρ_l, and the volumetric ratio of spiral reinforcement, ρ_s, were 1.18% and 0.61%, respectively. Reinforcing bars with the yield strength of 420 MPa (Grade 60) were used for both longitudinal and spiral reinforcement.

The lateral load capacity of the column was 1.29 MN. The computed yield and the ultimate displacements were 0.11 m and 0.58 m, respectively, and thus the ductility capacity of the column was 5.2.

The reinforced concrete bridge column was idealized as a two-dimensional discrete model, as shown in **Fig. 1**.

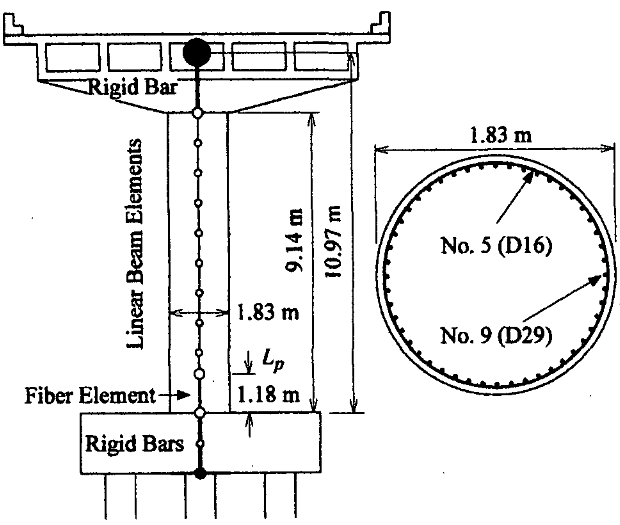
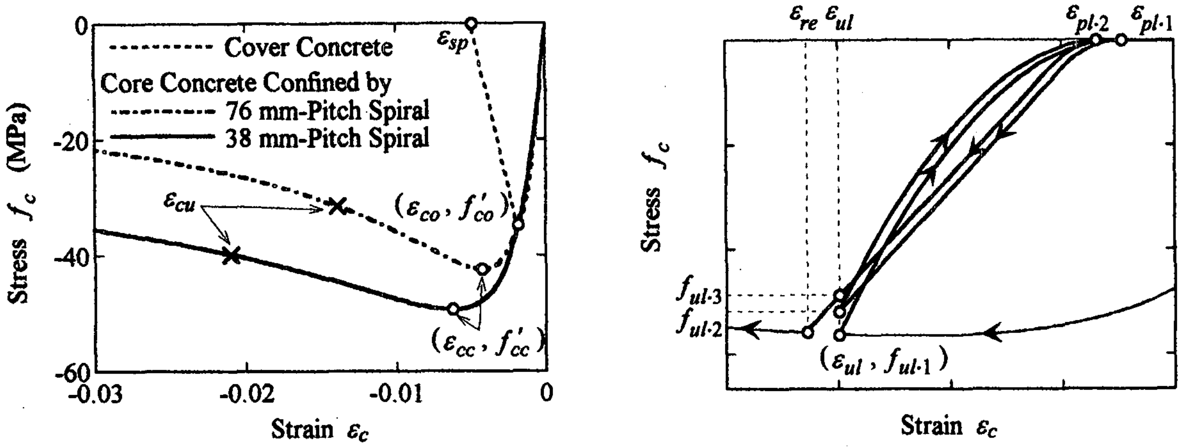
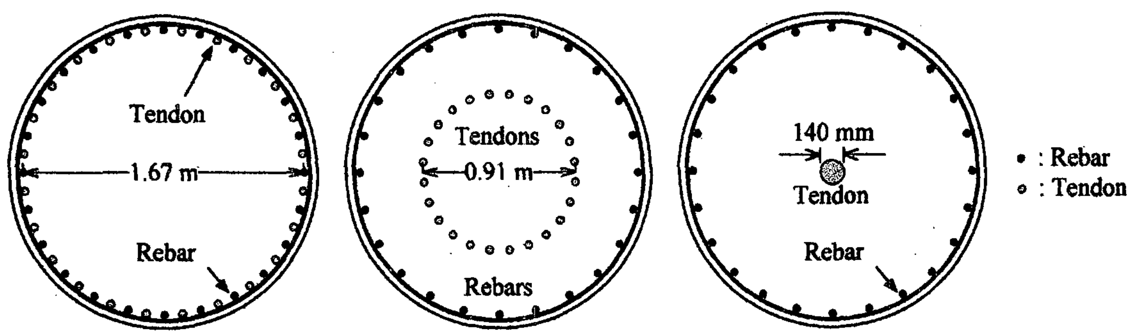


Fig. 1 Bridge Column Analyzed



(a) Envelope Curves (b) Unloading and Reloading Paths

Fig. 2 Concrete Stress-Strain Models



(a) Column PC-A (b) Column PC-B (c) Column PC-C

Fig. 3 Cross Sections of Columns with Tendons

The flexural hysteretic behavior of the plastic hinge region was idealized by fiber elements. The plastic hinge length was assumed to be 1.18 m. Rigid bars were used to model the footing and the element from the top of the column to the gravity center of the superstructure. Linear beam elements with cracked stiffness properties were used for the remainder of the column.

For the quasi-static analyses, predetermined cycles of displacement were imposed at the gravity center of the superstructure. The amplitude in the first cycle was 0.127 m, which was almost the same displacement as the yield displacement of the column. The lateral displacement was increased step wise up to 0.635 m, which was little over the estimated ultimate displacement of the column.

Figure 2 shows the stress-strain model of concrete for the fiber elements. The confinement effect on concrete properties and the stress-strain envelope curve were evaluated based on the Mander model[5]. The peak stress of core concrete, f'_{cc}, the strain at the peak, ε_{cc}, and the ultimate strain, ε_{cu}, were 0.0043, 42.4 MPa and 0.014, respectively. The descending branch of cover concrete was idealized as a linear function. Tensile strength of concrete was disregarded in this study. The model proposed by Sakai and Kawashima[6] was used for unloading and reloading paths.

The envelope curve of the reinforcing steel was idealized as a bilinear model, with the yield strength and the strain-hardening ratio equal to 414 MPa and 2%, respectively. To represent the hysteretic behavior of steel, the modified Menegotto-Pinto model[7] was used.

3. RC COLUMN WITH PRESTRESSED TENDONS

First, a series of quasi-static cyclic analyses on the reinforced concrete bridge column was conducted to study the effects of magnitude of axial load and amount of longitudinal reinforcement on the hysteretic behavior of the column. Based on the results, it was concluded it is possible to reduce residual displacement following an inelastic deformation by reducing the amount of longitudinal reinforcement and adding axial load. Therefore, it might be effective to replace some reinforcing bars with prestressed tendons for mitigation of residual displacements of RC columns.

To further understand the effect of using prestressed

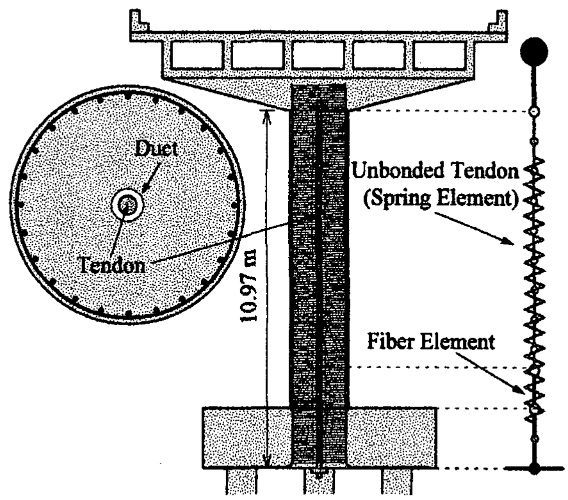


Fig. 4 Re-Centering RC Column and Its Analytical Model

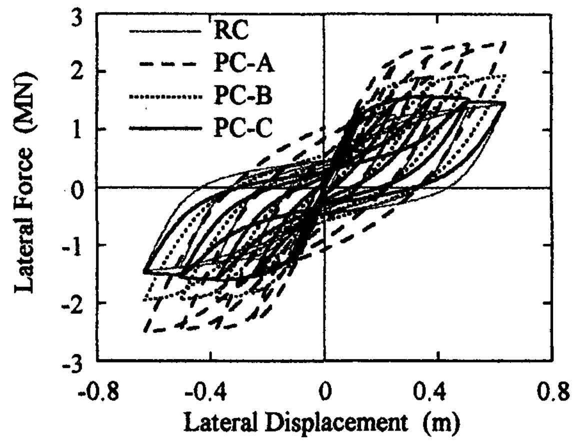


Fig. 5 Hysteresis of Columns with Prestressed Tendons

tendons, a series of quasi-static cyclic analyses was conducted for columns with different configurations of tendons as shown in **Fig. 3**. The configuration of the tendons was set up as follows: First, half of the rebar was replaced with prestressed tendons and the diameter of the configuration of tendons changed to 1.67 m to 0 m; the total area of tendons was kept constant at an equivalent area of 24 No. 9 bars. Column PC-A had half of its rebar replaced by 29 mm-diameter tendons. Thus, the diameter of the configuration of the tendons was 1.67 m, replicating the diameter configuration of the rebar. In Column PC-B, the configuration of tendons was 0.91 m in diameter. Column PC-C had one tendon, with an effective diameter of 140 mm, located at the center of the column. Note that Column PC-A and PC-B presented a challenge because it required a great deal of time and labor to prestress twenty-four tendons.

Grade 270 strand was used for the prestressed tendons. The elastic modulus of the strand E_{ps}, the essentially elastic strain $\varepsilon_{ps,EE}$, and the ultimate strain $\varepsilon_{ps,u}$ and strength $f_{ps,u}$ were 196.5 GPa, 0.0086, 0.03 and 1860 MPa, respectively. The total prestressing force was assumed to be 4.5 MN, resulting in a total axial load ratio of 10%. The stress induced in the tendon by 4.5 MN prestressing force was 293 MPa, which was only 16% of the ultimate strength of the Grade 270 strand.

To prevent undesirable premature crushing of concrete due to the additional axial load by the prestressed tendons, it was necessary to provide additional confinement for the column. To enhance the confinement of the core concrete, the spiral pitch was reduced from 76 mm to 38 mm. Accordingly, the denser spirals increased ρ_s up to 1.22%. ε_{cc}, f'_{cc} and ε_{cu} of the core concrete confined by 38 mm-pitch spirals were 0.0063, 49.3 MPa and 0.021, respectively, as shown in **Fig. 2**.

Yielding of the tendons was another concern. Tendons have relatively limited ability to deform inelastically, and yielding would reduce the effective prestressing force applied to the column during subsequent cycles. Therefore, tendon strains were reduced for many of the post-tensioned columns considered by unbonding the tendons from the concrete by means of ducts or some debonding medium. The ducts considered in this study to debond tendons were provided from the bottom of the footing to the top of the column, as shown in **Fig. 4**. The unbonded length was 10.97 m; six times that of the column diameter. The column with the unbonded center tendon and with the denser spirals is referred to later in this study as the Re-Centering RC Column.

To idealize columns with bonded tendons, fibers including the tendon's properties were added in the fiber elements. Unbonded tendons, however, were idealized by spring elements, as shown in **Fig. 4**. The stress-strain envelope curve of the tendon was idealized as a bilinear model, with the yield strength $f_{ps,y}$ = 1800 MPa, and the strain-hardening ratio R_{ps} = 2%; the modified MP model was used for unloading and reloading paths.

4. EFFECT OF PRESTRESSED TENDONS

(1) Effect of Configuration of Tendons

Figure 5 compares the force versus displacement hysteresis between the RC column and three columns containing the different tendon configurations. Tendons were bonded to the concrete and no additional confine-

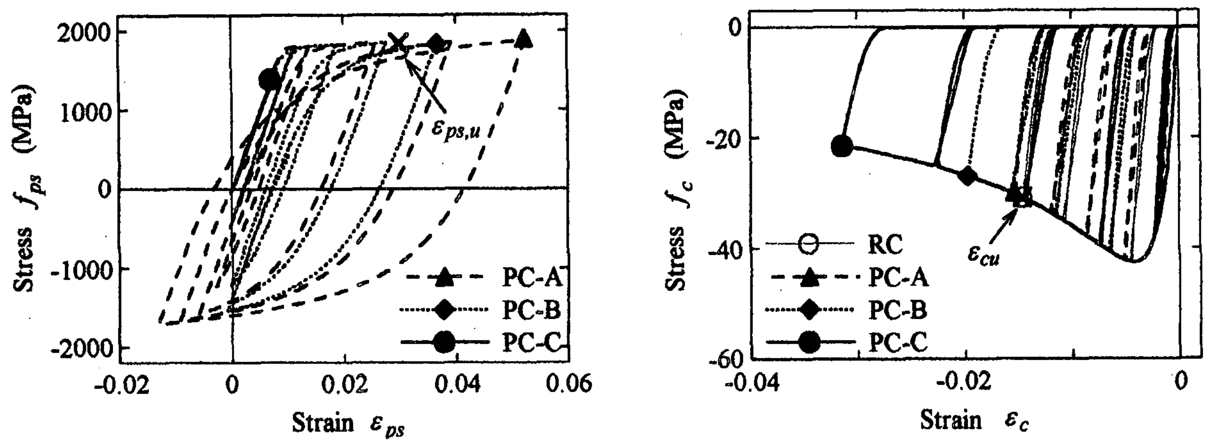


Fig. 6 Stress vs. Strain Hysteresis of Columns with Prestressed Tendons

ment was provided for these columns.

In the simple configuration where half of rebar was replaced by tendons (Column PC-A), the lateral strength went up to 2.52 MN; this was 74% greater than that of the RC column. Such high flexural strength may be undesirable in capacity design due to the increased shear capacity required for the column, and larger design forces needed for the superstructure, footing and piles.

When the tendon was concentrated at the center of Column PC-C column, the lateral force reached the peak strength (1.6 MN and 10% larger than the RC column) at the lateral displacement of 0.38 m, and then the lateral force decreased gradually as the lateral displacement increased.

Incorporating prestressed tendons resulted in smaller residual displacement upon unloading from the peak displacement reached during a cycle, but the configuration of the bonded tendons had little significant effect on the amount of reduction. The residual displacements of all three prestressed columns were approximately 25% smaller than that of the RC column.

Figure 6 compares stress versus strain hysteresis of a tendon and the core concrete at the compressive edge; the hysteresis of the tendon at the tension-most tendon is presented for Columns PC-A and PC-B, while the hysteresis of the center tendon is presented for Column PC-C. Strain induced in the tension-most tendon in Columns PC-A and PC-B exceeded 3%, the ultimate strain of the tendon $\varepsilon_{ps,u}$. This is a critical problem, potentially causing fracture of the tendons and significant loss of the lateral force carrying capacity of the column. Although the tendon at the center of Column PC-C did not yield, the maximum strain induced was 0.0074, which was nearly $\varepsilon_{ps,EE}$. The core concrete strain computed for Column PC-C was more than twice the estimated ultimate strain.

(2) Effect of Additional Confinement of Concrete and Unbonding of Tendons

Based on the previous results, it was decided that the prestressed columns needed additional confinement for the core concrete and that the tendons should be unbonded. Because Column PC-C shows better performance, i.e., desirable flexural strength, ease of construction and smaller tendon strain, results presented hereinafter will focus on Column PC-C with an unbonded center tendon and with 38 mm-pitch spirals. This column is referred as the Re-Centering RC column.

Figure 7 shows hysteretic behavior predicted for the Re-Centering RC Column. The stiffness of the Re-Centering RC Column changed sharply when the rebar began yielding in tension. After yielding, the force steadily increased with the positive post-yield stiffness, reaching a maximum strength of 1.44 MN: the skeleton curve was almost the same as that of the RC column. Unbonding of the tendon significantly affected the residual displacement upon unloading from a peak displacement. The residual displacement in the fifth cycle was 0.061 m, which was only 14% of that of the RC column.

The peak strain of the tendon was only 0.0035, 40% of $\varepsilon_{ps,EE}$ when it was unbonded. Furthermore, the maximum core concrete strain was 0.018, which was 14% smaller than the ultimate strain ε_{cu}.

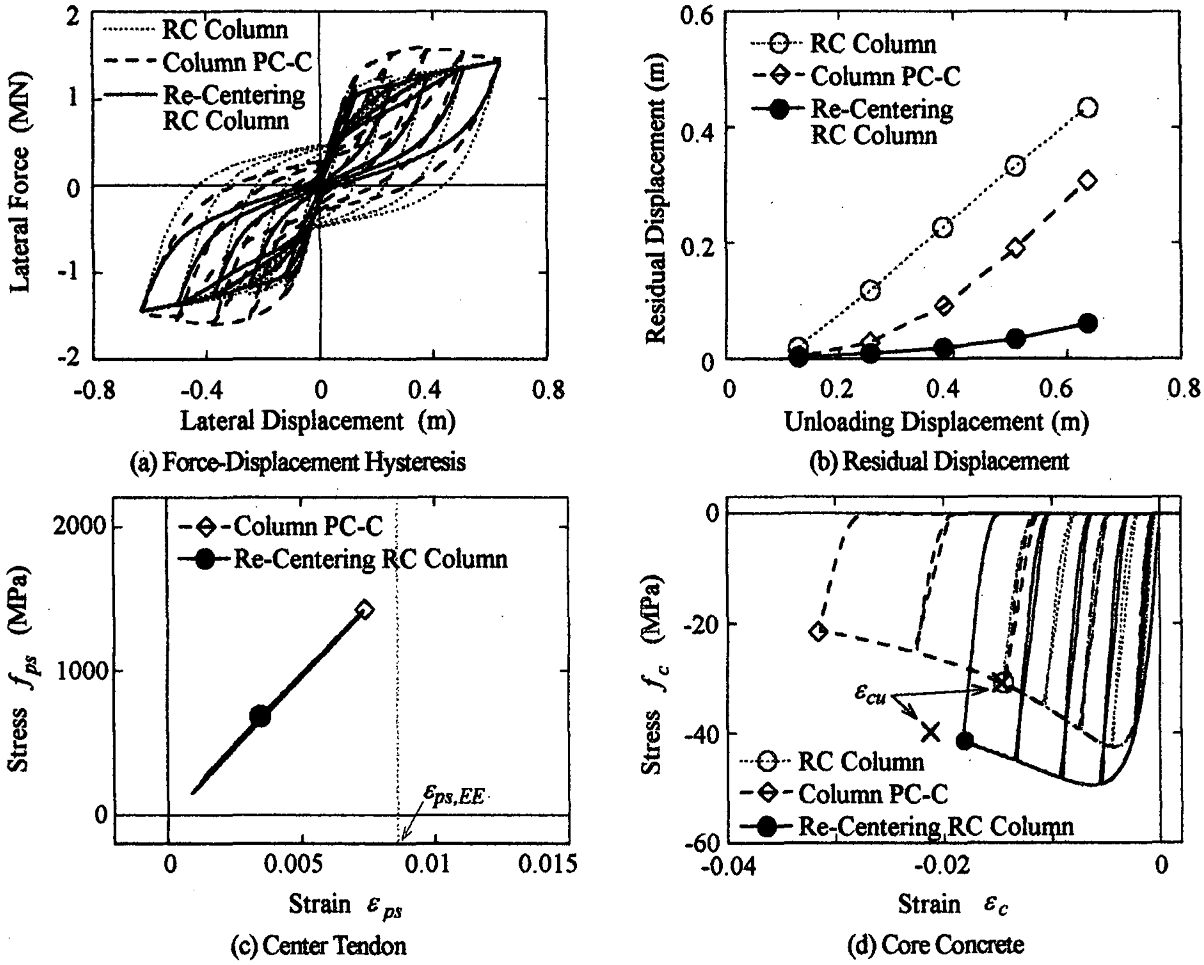


Fig. 7. Hysteretic Behavior of Re-Centering RC Column

Table 1 Near-Field Earthquake Ground Motions from SAC

Record	Earthquake	M	$\varDelta$	PGA
Tabas	Tabas, Iran, 1978	7.4	1.2	8.83
Los Gatos	Loma Prieta, USA, 1989	7.0	3.5	7.04
Lexington Dam	Loma Prieta, USA, 1989	7.0	6.3	6.73
Petrolia	Cape Mendocino, USA, 1992	7.1	8.5	6.26
Erzincan	Erzincan, Turkey, 1992	6.7	2.0	4.24
Landers	Landers, USA, 1992	7.3	1.1	7.00
Rinaldi	Northridge, USA, 1994	6.7	7.5	8.73
Olive View	Northridge, USA, 1994	6.7	6.4	7.18
JMA Kobe	Hyogo-ken Nanbu, Japan, 1995	6.9	3.4	10.67
Takatori	Hyogo-ken Nanbu, Japan, 1995	6.9	4.3	7.71

M : Magnitude, $\varDelta$: Epicentral Distance (km)
PGA: Peak Ground Acceleration (m/sec^2)

5. DYNAMIC RESPONSE OF RE-CENTERING RC COLUMN

The two-dimensional models shown in **Figs. 1 and 4** were used to carry out dynamic analyses of the columns. The models were fixed at the bottom of the footing and the soil-structure interaction was disregarded. The natural period of the 1st mode was 1.30 seconds based on the Eigen-value analysis for the model with cracked stiffness properties of the column. The SAC impulsive near-field ground motions[8], which are shown in **Table 1**, were used for the dynamic analyses.

Figure 8 compares the dynamic response of the Re-Centering RC Column and the RC column subjected to the Lexington Dam record obtained during the 1989 Loma Prieta earthquake. The maximum response displacement of the Re-Centering RC Column was almost the same as that of the RC column. The residual displacement of the Re-Centering RC Column was only 0.0013 m while that of the RC column was 0.042 m.

Figure 9 compares the maximum response and the residual displacements for the ten ground motions. As a whole, the maximum response displacements of the Re-Centering RC Column were almost the same as those of the RC column. When the columns were subjected to the Los Gatos Record or the Takatori record, the response exceeded the ultimate displacement. The residual displacements of the Re-Centering RC Column were only about 10% of those of the RC column.

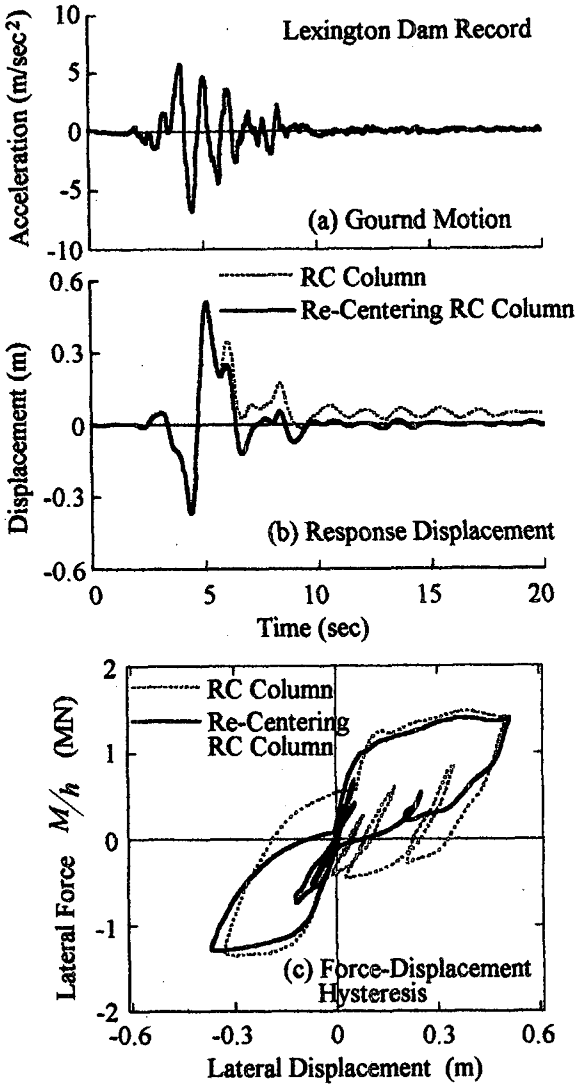


Fig. 8 Dynamic Response of Columns

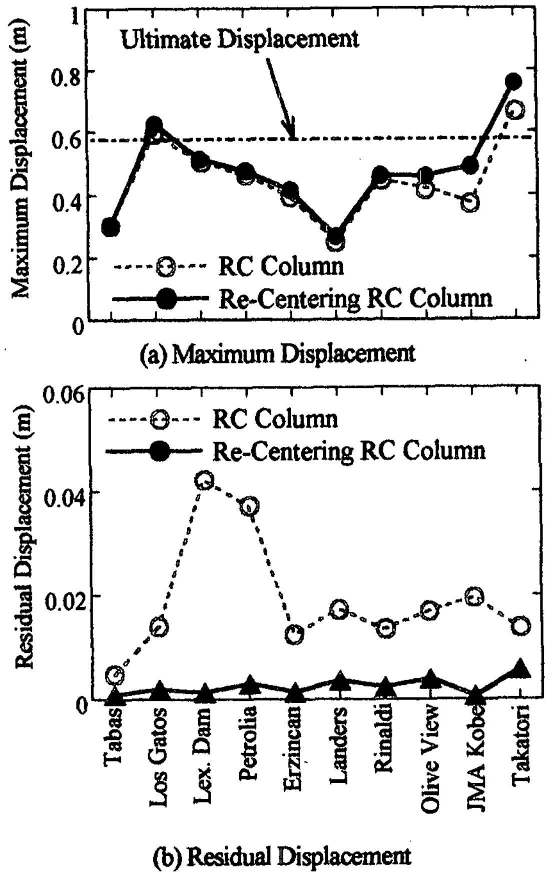


Fig. 9 Maximum and Residual Displacement

6. CONCLUSIONS

1) Replacing half of the rebar with tendons and applying an axial load that was equivalent to the axial load due to the dead load as the prestressing force resulted in a 25% decrease in the residual displacement upon quasi-static unloading from a peak inelastic displacement compared to the RC column.

2) When the center tendon in Column PC-C column was unbonded, the residual displacement in the fifth cycle was 0.061 m (which was 86% smaller than that of the RC column), and desirable flexural strength was obtained. Furthermore, unbonding the tendon decreased the core concrete strain and did not exceed the ultimate strain if the additional confinement was provided.

3) The column with the unbonded center tendon (Re-Centering RC Column) performed very well under strong ground shaking. The residual displacements of the Re-Centering RC Column were only about 10% of those of the RC column, while the maximum response displacements of the Re-Centering RC Column were virtually the same as those of the RC column.

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