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# EXPERIMENTAL EVALUATION OF THE RESIDUAL AXIAL LOAD CAPACITY OF CIRCULAR BRIDGE COLUMNS

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**ABSTRACT :**

Evaluation of the post-earthquake capacity of a bridge to carry self-weight and traffic loads is essential for a safe and timely re-opening of the bridge after an earthquake. The ability of a bridge to function depends directly on the remaining capacity of the bridge columns to carry load. An experimental study was conducted to investigate the relationship between earthquake-induced damage in reinforced concrete bridge columns and the capacity of the columns to carry axial load in a damaged condition. Four essentially identical one-fourth scale models of the prototype columns are designed for testing in the cantilever configuration, assuming an inflection point at the column mid-height. A two-stage testing procedure is used. In the first stage, three columns were tested under lateral load by applying a bi-directional quasi-static incremental lateral displacement protocol with circular orbits of displacement up to the predetermined, incrementally increasing displacement ductility targets: 1.5, 3 and 4.5. An axial load equal to 10% of the column's nominal axial load capacity is maintained during lateral test. At the end of the lateral tests the columns are re-centered by cycling them at very low amplitudes of displacement. In the second stage, an undamaged and the three damaged columns are subjected to a monotonically increasing axial force up to the failure of the column. The residual axial load capacity of the damaged columns without presence of residual lateral displacement is then determined. The test results show a loss of up to 22% in axial load carrying capacity with increasing displacement ductility demand.

**KEYWORDS:** Bridge columns, axial load capacity, experiment, bridges, earthquake

## 1. INTRODUCTION

Over the past three decades significant progress has been made toward preventing the brittle failure modes in columns. Unless complete failure occurs, the columns subjected to earthquake excitation are expected to have some level of residual strength. Quantifying this level of residual strength has seen little treatment in research literature. Some researchers (e.g., Elwood, 2002) looked at the issue of axial failure, defined as the lateral demand at which a column fails to carry its prescribed dead load. Testing setups include constant axial loads applied during lateral excitation (e.g. Kato and Ohnishi, 2002, Yoshimura and Nakamura, 2002), as well as several axial load control strategies based on levels of lateral deformation (e.g., Tasai, 1999).

This study attempts to further the understanding of residual axial strength through an analytical and experimental approach. Four scaled models of typical circular bridge columns are tested in two phases. The first phase involves lateral cyclic displacement-controlled loading history up to a pre-determined level of lateral displacement ductility. The second phase involves compressing the damaged specimen by axial loading using a compression-tension machine. Such loading allows the determination of the residual axial load capacity after a controlled amount of lateral load-induced damage. These tests are used to develop the axial load capacity vs. ductility demand degradation curves and to calibrate the finite element model. The calibrated finite element model will be used to vary design parameters of the column and generate axial load degradation curves for a dense parameter space.

Two additional hybrid simulation tests, conducted at the *nees@berkeley* NEES Equipment Site, will be used to validate the proposed procedure. In the hybrid tests, the column specimens are subjected to earthquake ground motions that equivalent counterpart components would experience in a real bridge, followed by the axial load tests in the compression-tension machine. The ultimate goal of this research is to capture the degradation of strength in analytical models that can be applied to more complex bridge and building structures in demand and capacity simulations within the PEER Center PBEE framework (Cornell and Krawinkler, 2000).

## 2. EXPERIMENTAL INVESTIGATION

The ¼- scale bridge column specimens are designed using a prototype Caltrans bridge (Ketchum et. al. 2004) shown in Figure 1. The two principal parameters that affect the remaining axial load capacity of bridge columns are column aspect ratio (H/D) and column shear strength or transverse reinforcement ratio (Mackie, 2004). The different possible values of these two parameters, bounded by the provisions of the Caltrans SDC, are investigated. The specimen is then designed (referred to here as the base column specimen) based on the bridge Type 11 (Figure 2) in Ketchum et. al., 2004. The base column specimen, typical for a tall column overpass bridge has an aspect ratio (L/D) of 4 and a ratio of transverse reinforcement ($\rho_t$) of 0.75%. Since the scaled models of the prototype columns are designed for testing in the cantilever configuration, assuming an inflection point at column mid-height, the aspect ratio of the bridge columns (H/D) is 8.

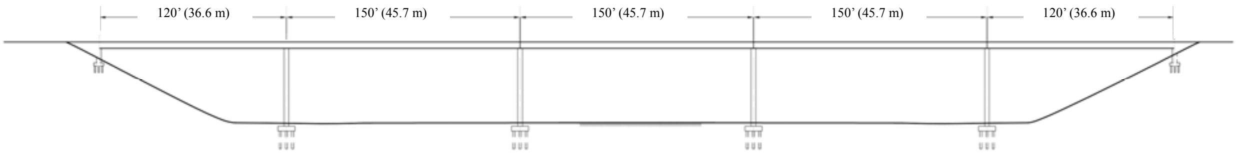



Figure 1 Prototype Caltrans bridge (Ketchum et. al., 2004)

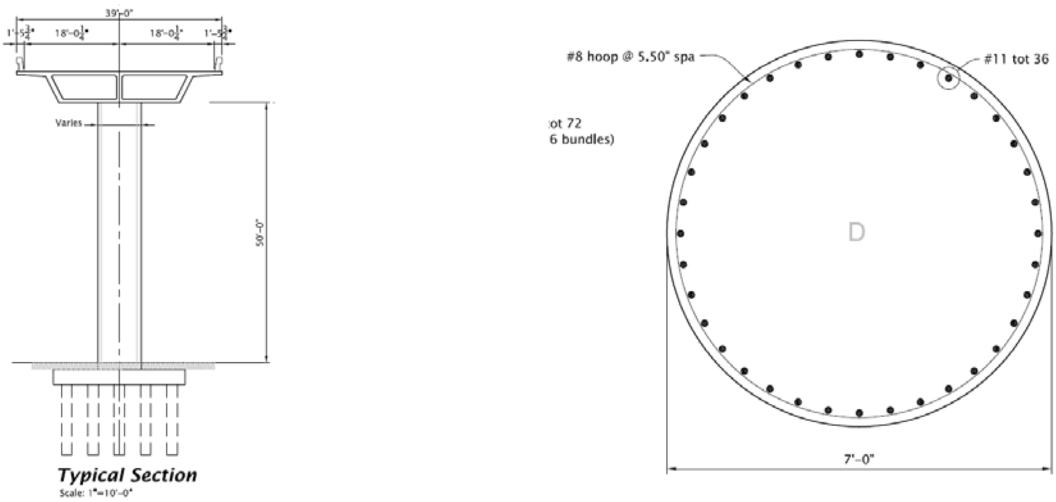
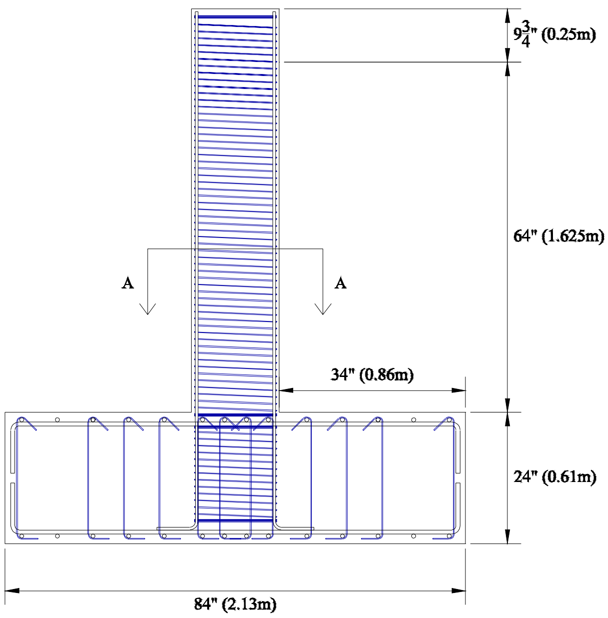
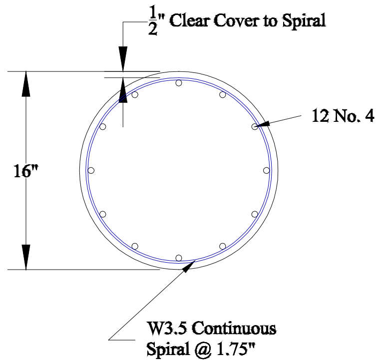



Figure 2 Bridge Column Type 11 (Ketchum et. al., 2004)

### *2.1. Geometry of the "Base" Column Specimen*

The specimen geometry and dimensions are detailed in Figure 3. The specimen is a 16-inch (0.4 m) diameter circular column, 64 inches (1.625 m) in height. The specified unconfined compressive strength of the concrete is 5 ksi (34.5 MPa). A Grade 60 steel (fy=60 ksi (420 MPa)) is used for the longitudinal reinforcement. The column has twelve longitudinal #4 (Ø13) reinforcing bars placed around its perimeter. The transverse steel reinforcement is W3.5 spiral with a center to center spacing of 1.25-inch (3.175 cm). The cover is 1/2" (1.3 cm) all around.




| | |
|---|---|
| Diameter | 16” (~0.4m) |
| Longitudinal reinforcement | 12#4 Gr60 ($\rho_l$=1.2%) |
| Transverse reinforcement | W3.5@1.25in ($\rho_t$ =0.75%) |
| Concrete | $f_c$‘=5ksi (~34.5MPa) |

Figure 3 Geometry and dimensions of base column specimen

## 2.2. *Loading Pattern*

In order to define the most suitable pattern of loading, nonlinear time history analysis was performed on the two models of existing bridges. The analysis is performed for 20 records of ground motion having different intensities: small, moderate and strong. The displacements at the top of the columns are traced and a circular loading pattern is chosen based on it. The circular loading pattern is defined by two cycles at each displacement level. In the first cycle, starting from the initial position O the specimen is displaced toward the position A, after which the circular pattern of displacement in clockwise direction follows until the end of circle, point B. The specimen is then moved back to the initial position O (red line in Figure 4). In the second cycle the path O-C-D-O is followed with the circular path C-D in the counterclockwise direction (blue line in Figure 4). The primary displacement levels are increased monotonically to provide an indication of the damage accumulation. The imposed displacement pattern of the two cycles at each displacement level provides an indication of the degradation characteristics.

For the test with displacement ductility target of 4.5, the increments in the magnitude of the displacement ductility are: 0.08, 0.2, 0.4, 1.0, 0.33, 1.5, 0.5, 2.0, 0.67, 3.0, 1.0, and 4.5. The pre-yield displacement levels include: a displacement level prior to cracking, two levels between cracking and yielding, and a level approximately corresponding to the first yield of the longitudinal reinforcement. For the post-yield displacement levels, the magnitude of the subsequent primary displacement level is determined by multiplying the current level by a factor ranging from 1.25 to 1.5. In the post-yield displacement history, each primary displacement level is followed by a small displacement level equal to one-third of the primary one. The displacement history is shown in Figure 5. The displacement histories for the lateral tests with displacement ductility targets of 1.5 and 3 are obtained by scaling the displacement history of the lateral test with displacement ductility target of 4.5 by 0.3 and 0.6, respectively.

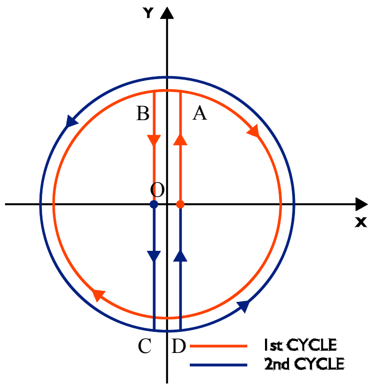



Figure 4 Loading Pattern

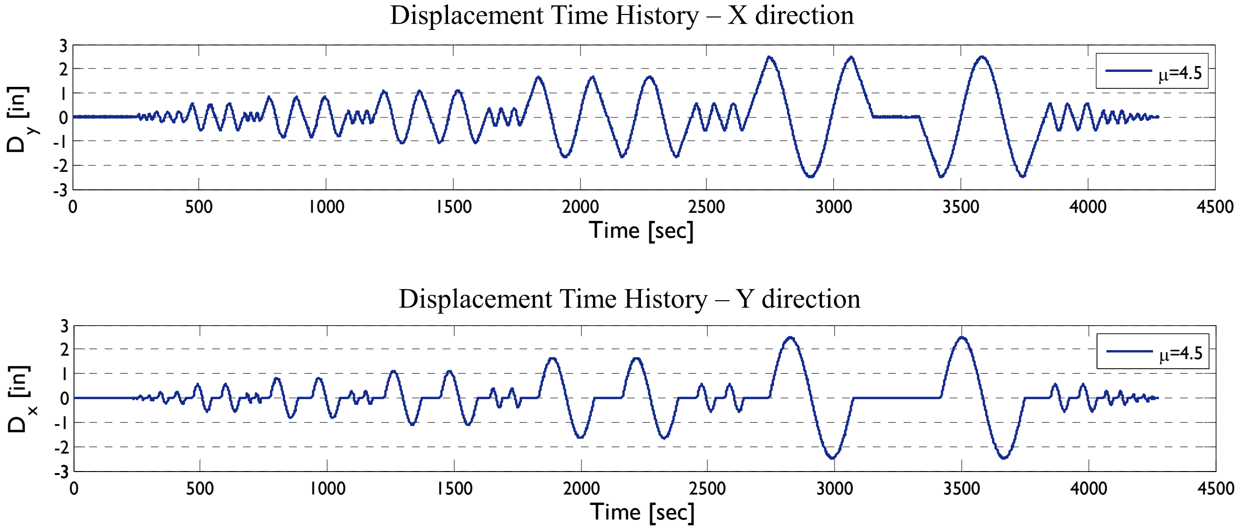


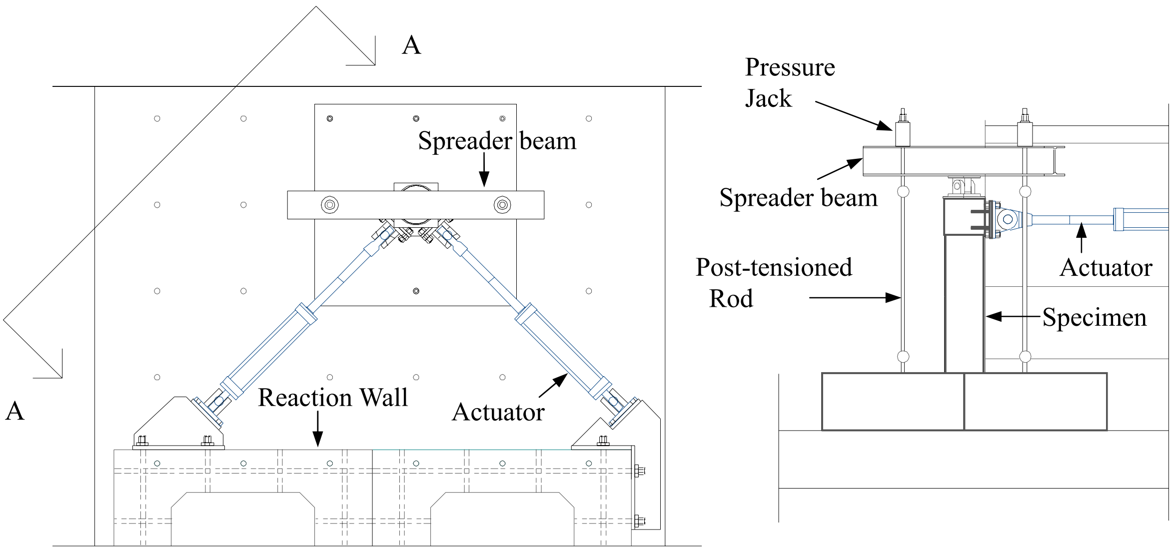

Figure 5 Displacement History (Displacement Ductility Target is 4.5)

## 2.3. *Lateral Test Setup*

In the first phase of the test, lateral and axial load are applied at the top of the column. The lateral cyclic load with circular orbits of displacement is applied using the two servo-controlled hydraulic actuators as shown in Figure 6. In order to attach actuators to the top of the column, a 0.5 inch (1.27 cm) thick steel jacket is embedded at the top of the reinforced concrete column during construction. The steel jacket has two vertical plates to provide attachment for the actuators and two horizontal plates to provide load transfer from the actuators to the column.

An axial load of 100 kips (445 kN) approximately equal to 10% of the column's nominal axial load capacity is maintained during the lateral test. This load represents the typical dead and live load carried by the column. The axial load is applied through a spreader beam using post-tensioning rods placed on either side of the column. Spherical hinges are provided at both ends of the rods to avoid bending of the rods during bi-directional displacements of the column. Moreover, a hinge connection is needed between the spreader beam and the column for the beam to remain horizontal in the plane of the rods during the lateral displacements of the column. In this way, buckling of the rods is also avoided.

In the second phase of the test, the three laterally damaged column specimens and one undamaged column specimen are compressed axially under a constant loading rate to induce axial failure of the columns. A 4,000,000 lb (2000 t) compression-tension machine located at the *nees@berkeley* Equipment Site (http://nees.berkeley.edu) was used.

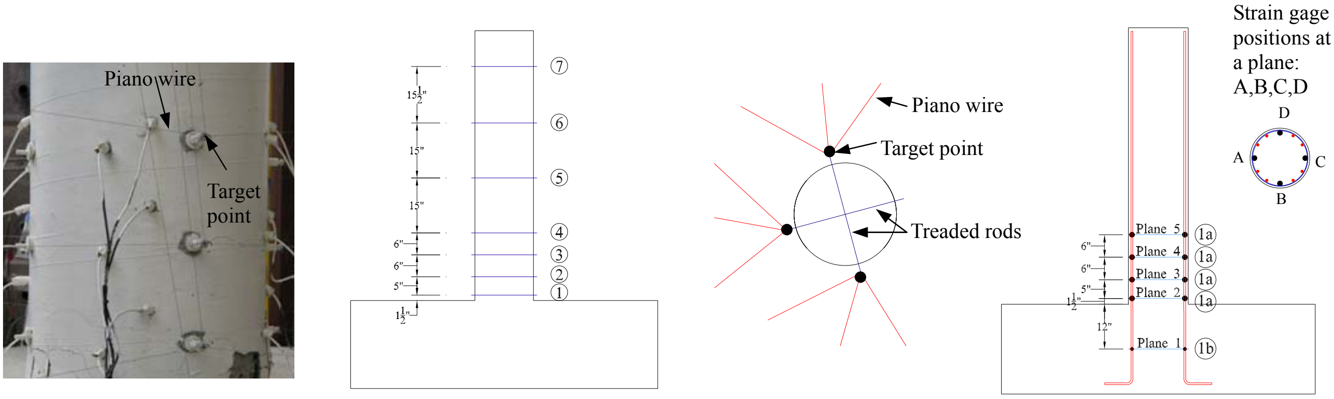



(a) Plain view (b) Elevation

Figure 6 Lateral Test Tetup

## 2.4. *Instrumentation*

The columns are instrumented externally using displacement potentiometers and internally using strain gages. At seven levels along the height of the column (Figure 7b), three points at each level (referred as target points, Figure 7c) are instrumented with three displacement potentiometers per point (Figure 7a and 7c). The instruments are connected to the target points of the column by piano wires. All instruments are attached to the instrumentation frames. There are three instrumentation frames encasing the column from three sides. The displacements of any target point at any level of the column are measured in three arbitrary spatial directions and mathematically transformed to displacements of the point in global coordinate system. The measured displacements of the three target points at one level are then used to derive the 6 degrees of freedom (3 displacements and 3 rotations) for the section at that level.



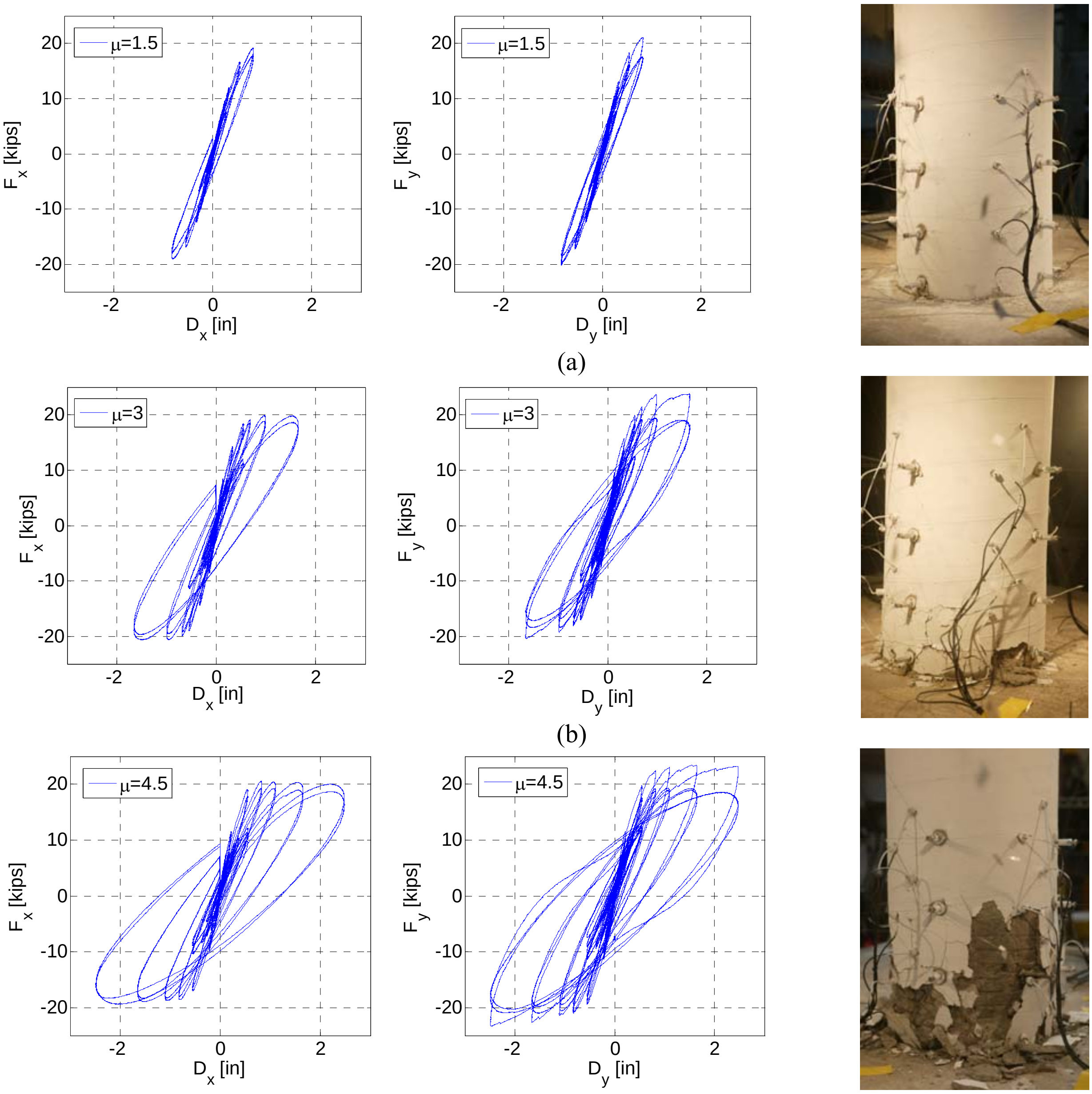

(a) (b) (c) (d)

Figure 7 Instrumentation: (a) Specimen Target Points with Their Piano Wires, (b) External Instrumentation Layout, (c) Layout of Target Points at a Level, (d) Internal Instrumentation Layout (strain gages)

Strain gages are attached to four out of twelve reinforcing bars at five levels along the height of the column (as labeled in the Figure 7d). The first level of strain gages is in the foundation zone (Plane 1) and the other four levels are in the plastic hinge region. The bars with the strain gages attached to it coincide with the axes of application of the load. The spiral reinforcement is instrumented by strain gages as well. The positions of strain gages attached to the spiral reinforcement coincide with the positions of the strain gages attached to the bars at levels 1a in Figure 7d.

## 3. TEST RESULTS

Figure 8 shows lateral force-displacement relationships in the principal axes of loading for the three lateral tests. Also shown are the lateral displacement ductility levels attained in each tests, and the column damage observed at the end of each lateral test.




(c)

Figure 8. Hysteresis Curves in X and Y Principal Directions and the Damage State of the Columns at the End of the Lateral Tests: (a) Displacement Ductility Target of 1.5, (b) Displacement Ductility Target of 3, (c) Displacement Ductility Target of 4.5

Figure 9 shows the results of the compression tests. Figure 9a shows the result of compression test on the laterally undamaged column specimen. This test provides the original axial strength of the column specimens. Figure 9b, c, d shows the results of compression tests on the laterally damaged column specimens. These tests provide the residual axial load carrying capacity of the column specimens after certain amount of lateral damage is induced in them.

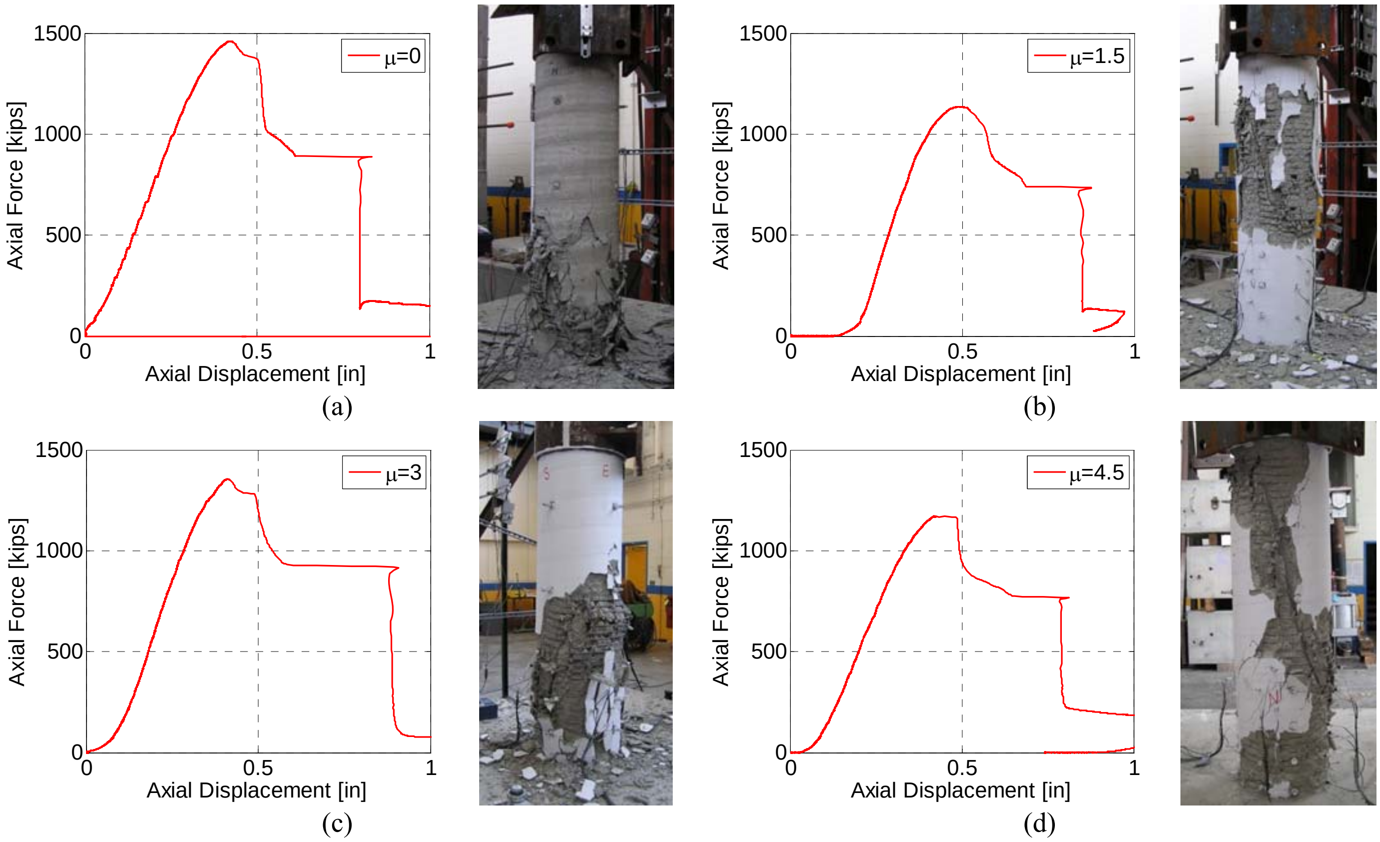



Figure 9 Axial Force-Axial Displacement Relationships and the Damaged State of the Column at the End of the Axial Tests: (a) Laterally Undamaged Column Specimen, (b), (c), (d) Laterally Damaged Column Specimens at Displacement Ductility Targets of 1.5, 3.0, and 4.5 Respectively

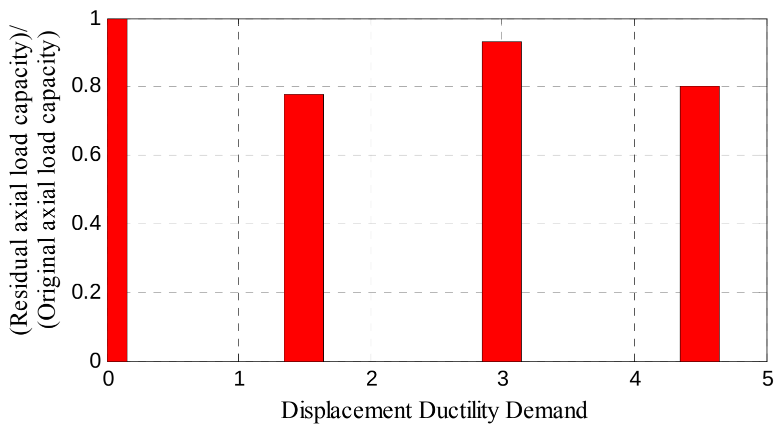



Figure 10 Residual axial load capacity vs. displacement ductility demand

The main objective of the experimental investigation is to express residual axial load carrying capacity of the bridge column as a function of displacement ductility demand. Based on the test results this relation is graphically presented in Figure 10. Although experimental procedure is developed to explore influence of pure material damage on the residual axial strength, the column with displacement ductility demand of 1.5 contained geometric imperfection of 0.5 inches (1.27 cm) at the top of the column. Due to the presence of non-zero lateral displacements at the top of the column during axial test, residual axial load capacity of this column is smaller then for the other columns. In order to explore the effect of geometric imperfections (residual displacements) on remaining axial strength of a bridge column after an earthquake, an analytical model will be developed.

## 4. CONCLUSIONS

Using the summary of the test results presented in Figure 10 it is concluded that the loss of axial load capacity dure to damage induced by lateral displacement does not exceed approximately 20% for displacement ductility demand levels smaller than 4.5. Thus, it can be concluded that the bridge columns designed by Caltrans SDC 1.3 (2004) will experience no significant loss of their nominal axial load carrying capacity due to an earthquake they are designed for. This conclusion is limited to the columns with no residual displacements after an earthquake. The effect of the residual displacements on the remaining axial load capacity of the bridge columns after an earthquake is yet to be determined.

## REFERENCES

Caltrans SDC (2004), Seismic Design Criteria, Version 1.3, California Department of Transportation, Sacramento, CA, USA, February 2004.

Cornell, A. and Krawinkler, H. (2000). Progress and Challenges in Seismic Performance Assessment. PEER Center News 3 (2), 1-2.

Elwood, K.J. (2002). Shake table tests and analytical studies on the gravity load collapse of reinforced concrete frames. PhD Dissertation, University of California, Berkeley.

Elwood, K.J. and Moehle, J.P. (2005). Axial Capacity Model for Shear-Damaged Columns. *ACI Structural Journal* **102:4**, 578-587.

Kato, D. and Ohnishi, K. (2002). Axial load carrying capacity of reinforced concrete columns under lateral load reversals. Technical report PEER 02, 247-255.

Ketchum, M., Chang, V. and Shantz, T. (2004). Influence of Design Ground Motion Level on Highway Bridge Costs. Technical Report PEER, 6D01.

Mackie, K. (2004). Degradation of reinforced concrete column axial strength. The 5th International Symposium in Civil Engineering 2004. Delft, Netherlands.

Mackie, K. and Stojadinovic, B. (2005). Fragility Basis for California Highway Overpass Bridge Decision Making. Technical Report PEER 02.

Shin, Y.B. and Moehle, J.P. (2002). Quarter-scale reinforced concrete specimen test. Internal technical report 2002, University of California, Berkeley.

Tasai, A. (1999). Residual axial capacity and restorability of reinforced concrete columns damaged due to earthquake. Technical Report PEER 10, 191-202.

Tasai, A. (2000). Residual axial capacity of reinforced concrete columns during shear degradation. Technical Report PEER 10, 257-267.

Yoshimura, M. and Nakamura, T. (2002). Axial collapse of reinforced concrete short columns. Technical Report PEER 21, 187-198