Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1156

CYCLIC BEHAVIOR OF PRECAST SEGMENTAL CONCRETE BRIDGE COLUMNS WITH HIGH PERFORMANCE STEEL REBAR AS ENERGY DISSIPATION BARS

Yu-Chen Ou[1], Mu-Sen Tsai[2], Kuo-Chun Chang[3], and George C. Lee[4]

ABSTRACT

The cyclic behavior of precast segmental concrete bridge columns with high performance (HP) steel rebar and that with conventional steel rebar as energy dissipation (ED) bars were investigated. The HP steel rebar is characterized by higher strength, greater ductility and superior corrosion resistance compared to the conventional steel rebar. Three large-scale columns were tested. One was designed with the HP ED bars and two with the conventional ED bars. The HP ED bars were fully bonded to the concrete. The conventional ED bars were fully bonded to the concrete for one column while unbonded for a length to delay fracture of the bars and to increase energy dissipation for the other column. Test results showed that the column with the HP ED bars had greater drift capacity, higher lateral strength and larger energy dissipation than that with fully-bonded conventional ED bars. The column with unbonded conventional ED bars achieved the same drift capacity and similar energy dissipation as that with the HP ED bars. All three columns showed good self-centering capability with residual drifts not greater than 0.4% drift.

Introduction

Cast-in-place construction of concrete bridges typically results in extensive damage to the on-site environment due to the need for land for construction activities. If the construction is in busy urban areas, it usually causes prolonged traffic disruption. Traffic disruption not only causes inconvenience to the traveling public but also increases the consumption of fuel due to detour and/or traffic jam and hence raises carbon dioxide emissions. Precast segmental construction by reducing on-site construction activities and time has been proven to be an effective method to address these issues (FHWA 2009). However, very few examples of bridges with precast segmental concrete columns can be found in regions of high seismicity such as west coast regions of the US, Japan, and Taiwan due to concerns on their seismic performance.

[1] Assistant Professor, Department of Construction Engineering, National Taiwan University of Science and Technology, Taipei, Taiwan.
[2] Ph.D. Student, Department of Civil Engineering, National Taiwan University, Taipei, Taiwan.
[3] Professor and Chairman, Department of Civil Engineering, National Taiwan University, Taipei, Taiwan.
[4] SUNY Distinguished Professor, Department of Civil, Structural and Environmental Engineering, University at Buffalo, Buffalo, U.S.A.

In a typical design of a precast segmental concrete bridge column, prestressing tendons throughout the column are stressed to apply compression force across precast joints, which with the compression force from gravity loads provide required flexural and shear strengths at the joints. Mild steel reinforcement is normally not continuous across the joints and thus contributes little to the strengths. It is used for positioning transverse reinforcement and for shrinkage and creep controls. Under lateral loads, the column shows a behavior same as a monolithic column prior to opening of the joints. Once the joints open, the column exhibits a nonlinear behavior with little energy dissipation but a small residual drift upon unloading (Hewes and Priestley 2002, Wang et al 2008, and Ou et al 2009). To increase energy dissipation, Wang et al. (2008) and Ou et al. (2009) propose to add mild steel reinforcing bars across the joints. The bars are referred to as energy dissipation (ED) bars to emphasize their function and to distinguish them from other mild steel bars that are not continuous across the joints. It has been shown that the use of the ED bars can significantly increase energy dissipation. However, when the columns are subjected to a large lateral displacement, significant joint opening will occur and likely cause premature fracture of the ED bars. Unbonding the bars for a length can decrease the strains and delay fracture. Alternatively, fracture can be delayed by using more ductile steel capable of absorbing greater energy before fracture. In this research, high performance (HP) steel rebar commercially known as Enduramet 32, designated as S24100 in ASTM A955 (ASTM 2004), is investigated for use as ED bars. It has superior ductility capacity than conventional carbon steel rebar. Additionally, it has excellent corrosion resistance (Schnell and Bergmann 2007). This can address potential corrosion problems resulting from opening of precast joints.

The energy dissipation and residual drift of a segmental column with ED bars increase as the strength contribution of the ED bars to the lateral strength of the column increases (Ou et al 2009). In Japan, it has been found difficult to re-center the superstructure of bridge columns with residual drifts exceeding 1/60 or with residual displacements more than 15 cm, whichever is smaller (Zatar and Mutsuyoshi 2002). As a result, the 1996 Japanese seismic design specifications of highway bridges requires that the residual drift developed at a bridge column after an earthquake not be greater than 1% (Kawashima 2000). Test results show that the segmental column can maintain a residual drift not greater than 1% provided the strength contribution of the ED bars is below approximately 35% of the lateral strength of the column (Ou et al 2009).

Three large-scale precast segmental concrete columns were tested in this research under lateral cyclic loading to demonstrate the cyclic performance of segmental columns with HP rebar as ED bars in comparison to conventional rebar.

Specimen Design and Test Setup

The design and test setup of the segmental columns tested are shown in Fig 1(a) and 1(b), respectively. Table 1 lists major design parameters. Each column consisted of four precast segments with a hollow cross section and one precast cap beam. Prestressing tendons were anchored at the underside of the foundation at one end and anchored on the top of the cap beam at the other end. The tendons were unbonded to the concrete and passed through ducts in the foundation, through hollow core of the segments and through ducts in the cap beam. Note that “unbonded” does not mean the tendons are “ungrouted” against corrosion. The tendons can be placed in smooth polyethylene pipes that are not bonded to concrete and fully grouted for corrosion protection. The tendons can also be epoxy coated for enhanced corrosion resistance.

The total prestressing force was 1042 kN, which was carried by four tendons, each consisting of two D15 seven-wire strands. The prestressing force was determined to ensure no opening of the precast joints under a lateral load corresponding to an assumed moderate earthquake. The prestress corresponded to 0.55 tendon yield stress. This is lower than typically used for prestressed concrete. Lower initial prestress and unbonding the tendons were intended to minimize yielding of the tendons and hence preserve an axial force necessary for self-centering capability. The specified gravity load was 1456 kN or 0.1 $f'_{co} A_g$, typical for a bridge column, where f'_{co} is the specified concrete strength, 27.6 MPa, and A_g is the gross cross-sectional area of the column. The gravity load was applied to the cap beam by two hydraulic actuators and remained constant throughout the testing. The displacement-controlled lateral cyclic loading was applied by a hydraulic actuator at one end anchored to the reaction wall and at the other end to the cap beam. The drift levels included 0.25%, 0.375%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0%, 4.0%, 5%, and 6% with each drift level repeated twice.

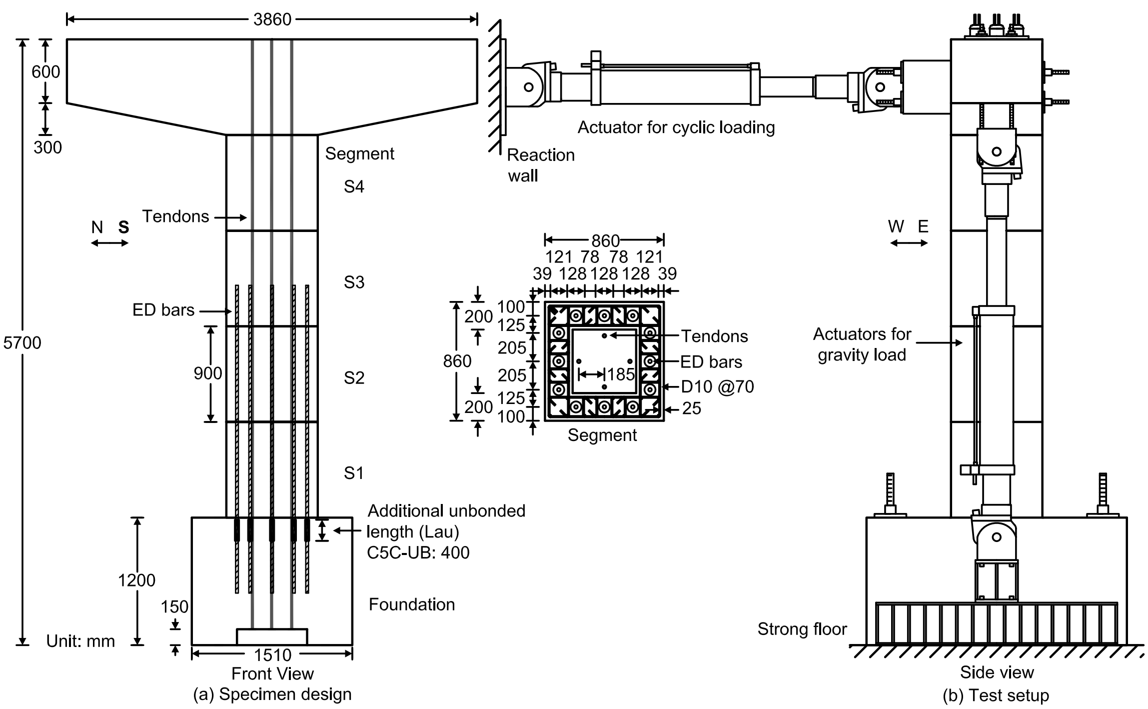


Figure 1. Specimen design and test setup

Table 1. Design parameters

Column	Gravity load (kN)	Prestressing force (kN)	ED bar Ratio (%)	L_{au} (mm)
C5C-FB	1456	1042	0.5	0
C5C-E32	1456	1042	0.5	0
C5C-UB	1456	1042	0.5	400

The conventional rebar used is low-alloy steel deformed bars conforming to ASTM A706 (ASTM 2004), typical for seismic design. Fig. 2 shows the monotonic tension responses of the conventional rebar and the HP rebar. The uniform elongation before necking of the HP rebar was 48%, more than three times that of the conventional rebar, 13%. In addition, the HP rebar has higher yield and ultimate strengths than the conventional rebar as listed in Table 2. The three columns were designed with 0.5% ED bar ratio. The ED bar contribution to the column lateral strength, denoted as λ_{ED} and defined by Eq. (1), was calculated to be lower than 35% for the three columns. The 35% limitation was to ensure that residual drifts would not exceed 1% before failure of the column as previously mentioned. The value of λ_{ED} for each column will be presented later using measured column strengths.

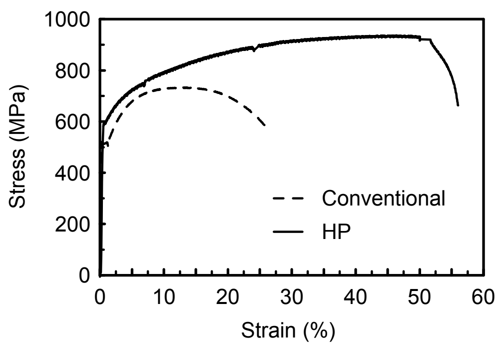


Figure 2. Monotonic tension behavior of conventional and HP ED bars

Table 2. Measured material properties

Concrete	Grout	Conventional rebar		HP rebar		Prestressing tendons	
Compressive strength (MPa)	Compressive strength (MPa)	Yield strength (MPa)	Peak strength (MPa)	Yield strength (MPa)	Peak strength (MPa)	Yield strength (MPa)	Peak strength (MPa)
45	49	454	665	580	939	1682	1852

$$\lambda_{ED} = \frac{V - V_0}{V} \tag{1}$$

where V =lateral strength of a column with ED bars; and V_0 =lateral strength of that column without considering the ED bars.

The ED bars were inserted through corrugated steel ducts during assembling of the columns. The ducts were grouted using a cement-based grout with an actual compression strength of 49 MPa. The ED bars were terminated in segment S3. This is because the moment demand at the joint between segments S3 and S4, denoted as joint S3-S4, at the peak lateral force of the column was found to be lower than the moment that would result in a compression depth lower than half the sectional diameter. The embedded lengths of the bars into segment S3 were determined by multiplying the development length calculated from AASHTO Section

5.11.2.1 (AASHTO 2007) by a ratio of the predicted bar stress at that joint to the ultimate stress of the bar. For the HP ED bars, the computed development length was further multiplied by 1.1 to take into account the higher ratio of the ultimate to yield strengths of the bars (Table 2). Columns C5C-FB and C5C-UB used the conventional rebar while column C5C-E32 used the HP rebar as ED bars. The ED bars of columns C5C-FB and C5C-E32 were fully-bonded, i.e., bonded to the concrete along their entire length while those of column C5C-UB were unbonded starting from joint foundation-S1 into the foundation for a length of 400 mm. The length is denoted as additionally unbonded length or L_{au}. The unbonding was done by wrapping the bars with duct tape. The 400-mm unbonded length was to ensure no fracture of the ED bars up to 5% drift.

Experimental Results

General observations

The relationships between lateral force and drift of the three columns are shown in Fig. 3. The lateral force was calculated from the force of the lateral actuator subtracted by the horizontal components of the forces of the two vertical actuators. Test results in terms of peak values are summarized in Table 3. For all three columns, flexural cracks first appeared on the east and west surfaces at 1.5% drift and then propagated towards the centroid as appeared on the north and south surfaces (Fig. 4) until 3% drift at which the lateral forces of the columns began to decrease due to the P-Δ effect. At the peak drifts during cyclic loading between 1.5% to 3% drifts, the widths of these cracks were measured to be between 0.06 and 0.15 mm, much smaller than the amount of corresponding joint opening that was on the order of 10 to 20 mm. Most of the cracks occurred on segment S1. Column C5C-E32 had a few cracks on segment S2 due to a higher lateral strength resulting from the use of the HP ED bars (Fig. 4(b)). All three columns failed due to fracture of the ED bars, which occurred at 3%, 6%, and 6% drifts for columns C5C-FB, C5C-E32, and C5C-UB, respectively. Thus, the drift capacities were set as 2%, 5%, and 5%, respectively. Note that the sudden drops of lateral force in Fig. 3(a) indicate occurrences of ED bar fracture. Spalling of the cover concrete only occurred on the compression faces around joint foundation-S1 as shown by the regions with black color and was found to be minor.

Opening of joint foundation-S1 was quite significant and increased approximately linearly with the increase of drift levels. Opening of joint S1-S2 was much smaller than that of joint foundation-S1 and saturated after the peak lateral force had been reached at 2% drift. Opening of the other joints were found to be negligible Fig. 5(a) shows column C5C-E32 at 6% drift. Large amount of opening of joint foundation-S1 and fracture of one of the ED bars can be clearly seen in Fig. 5(b).

Table 3 Test results

Column	Lateral strength (kN)	Ultimate drift (%)	λ_{ED} (%)	Max ζ_{eq} (%)	Max residual (%)
C5C-FB	370	2.0	25	10	0.1
C5C-E32	386	5.0	28	15	0.4
C5C-UB	363	5.0	23	16	0.4

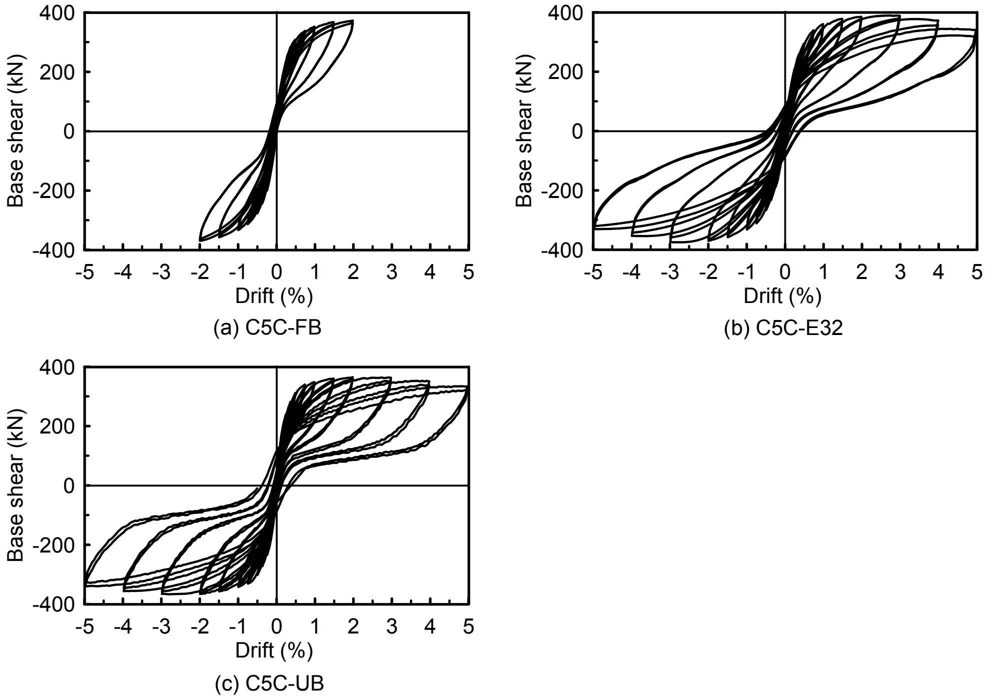


Figure 3 Lateral force versus drift

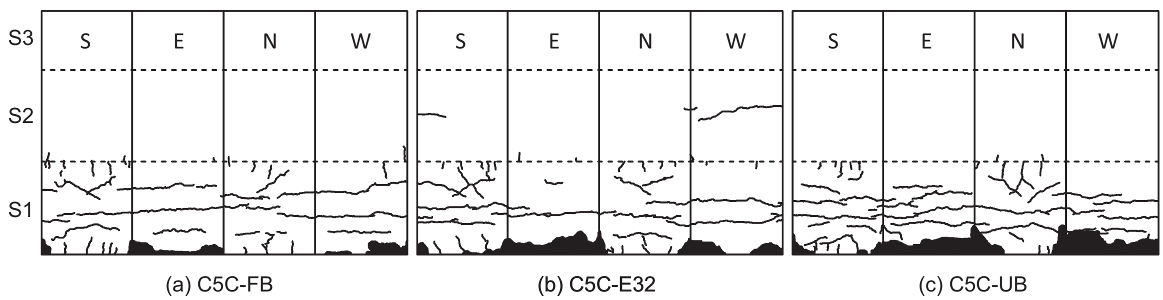
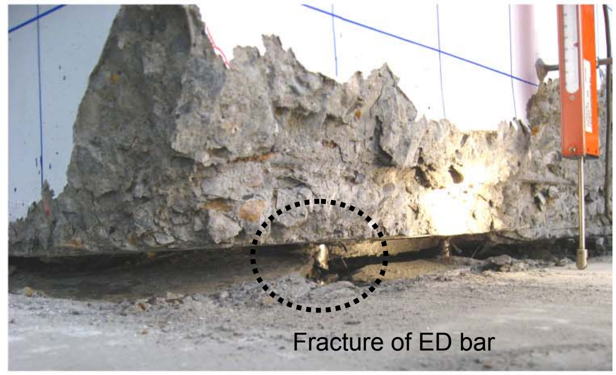


Figure 4. Cracks and spalling of concrete

(a) North-east view of the column



(b) Close view of joint foundation-S1

Figure 5.   Column C5C-E32 at 6% drift

Performance of HP ED bars and effects of unbonding

It can be seen from Fig. 6 that for a given drift, the conventional ED bar and HP ED bar of columns C5C-FB and C5C-E32, respectively, were subjected to a similar strain. For example, both were subjected to a strain of approximately 2.5% at 1% drift. For column C5C-FB, six of the critical ED bars, which were located at the east and west walls at joint foundation-S1 of the column fractured at 3% drift. The fracture caused a significant drop in the lateral force as shown in Fig. 3(a). For column C5C-E32, the HP ED bars were able to sustain cyclic loading up to 5% drift of the column without any fracture. Compared to column C5C-FB, the drift capacity of column C5C-E32 was greatly increased from 2% to 5% and hence the maximum energy dissipation in terms of equivalent viscous damping ratio ζ_{eq} was improved from 10% to 15% (Table 3). Additionally, column C5C-E32 showed a higher lateral strength. These demonstrate the capability of the HP ED bar to increase the ductility, energy dissipation and lateral strength of a segmental column. Instead of using the HP bars, fracture of the conventional ED bars can be delayed by unbonding the bars for a length. By comparing the ED bar strains of columns C5C-FB and C5C-UB as illustrated in Figs 6(a) and 6(c), respectively, it can be seen that unbonding for 400 mm effectively decreased the ED bar strain. For example, at 0.5% drift, the maximum ED bar strains are 1.2% and 0.26% for columns C5C-FB and C5C-UB, respectively. Unbonding decreased the rate of increase of lateral force as expected but had little effects on the lateral strength (peak lateral force) (Table 3). It was because with such an unbonded length, the strains of the ED bars at the peak force of the column still well exceeded the yield strain. The corresponding stresses between the two columns did not vary to an extent that could cause a significant difference in the column lateral strengths. Unbonding slightly decreased energy dissipation for a given drift (Fig. 7) but increased maximum energy dissipation by delaying fracture of the bars. The column with unbonding achieved similar performance to that with the HP ED bar in terms of ductility and energy dissipation (Table 3 and Fig. 7). However, unbonding required additional labor work. In addition, unlike the prestressing tendons, which are fully unbonded and hence can be replaced if corrosion occurs, it is difficult to replace the ED bars. Better corrosion resistance is expected for the column with the HP ED bars than with unbonded conventional ED bars, because not only the HP ED bars have superior corrosion resistance but they are also fully grouted, which further enhances corrosion protection.

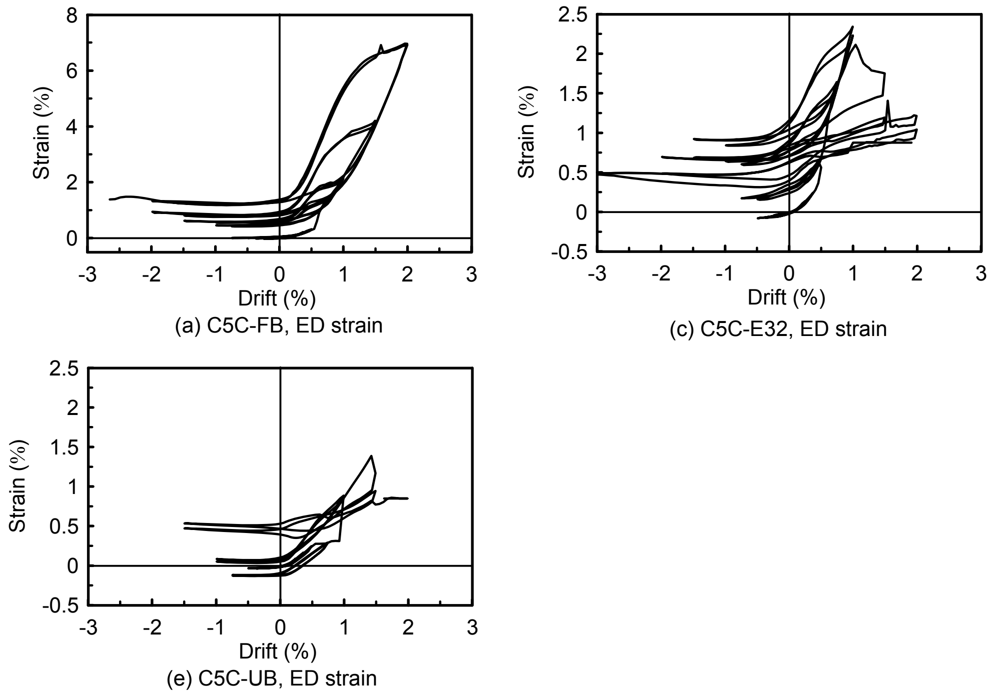


Figure 6. ED bar strain

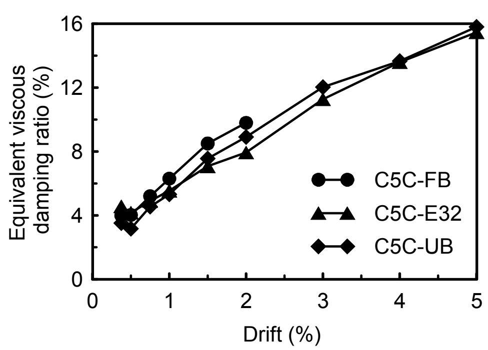


Figure 7. Equivalent viscous damping ratios ζ_{eq}

Self-centering capability

All three columns showed good self-centering capability with residual drifts not greater than 0.4% (Table 3 and Fig. 8). The columns exhibited a similar pattern of residual drift histories. The residual drift of column C5C-FB prior to failure was only 0.1%, much lower than the other two columns since it failed at a lower drift. The values of λ_{ED} of the three columns calculated from measured lateral strengths ranged from 23% to 28% (Table 3). The value of V_0 in Eq. (1) was equal to 278 kN and was obtained by testing of a corresponding column without ED bars (Ou et al 2009). As previously mentioned, λ_{ED} was limited to 35% to ensure a residual drift smaller than 1%. In other words, the lateral strength contribution from axial forces including gravity loads and prestressing forces needs to be greater than 65%. In contrast, lateral strength contribution from axial forces, typically only from gravity loads, in a conventional concrete bridge column normally ranges from 20% to 40%. This means a large residual drift may result for a conventional column after large earthquakes. However, for a segmental column with λ_{ED} limited to 35% to achieve the same lateral strength and drift capacity as a conventional column, the size and/or compressive strength of concrete cross section may need to be increased to accommodate additional prestressing forces.

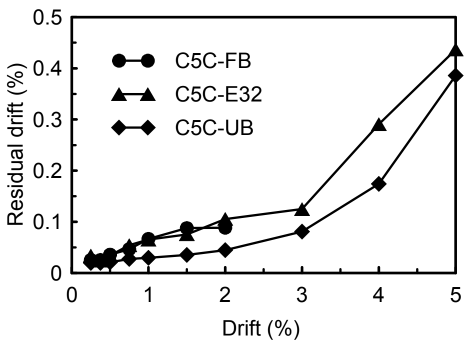


Figure 8. Residual drifts

Conclusions

The cyclic behaviors were investigated for three large-scale precast segmental concrete bridge columns designed with fully-bonded conventional ED bars, with fully-bonded HP ED bars, and with conventional ED bars that were unbonded for a length. Important conclusions are summarized as follows.

(1) All three columns failed due to fracture of the ED bars. The column with the HP ED bars showed higher drift capacity, greater energy dissipation, and higher lateral strength than that with fully-bonded conventional ED bars. Uunbonding the conventional ED bars for a length of 400 mm effectively delayed fracture of the bars while slightly decreased the lateral strength. The column with such unbonded bars achieved the same drift capacity and similar energy dissipation capability as that with the HP bars. However, unbonding requires considerable labor work and weakens corrosion protection of the bars.
(2) At the ultimate state, the extent of cracking and crushing of concrete was minor. Most of the column flexural rotations resulted from rotations of joints foundation-S1 and S1-S2. Rotations of the other joints and the segment bodies were found to be negligible.
(3) All three columns showed good self-centering capability with residual drifts not greater than 0.4%. The measured strength contributions of the ED bars to the lateral strengths of the three columns ranged from 23% to 28%.

Acknowledgements

This cooperative research is funded by the National Center for Research on Earthquake Engineering (NCREE) on the Taiwan side and by the Federal Highway Administration (FHWA) (Grant DTFH61-98-C-00094) on the U.S. side. Their support is gratefully acknowledged.

References

FHWA. 2009. Prefabricated bridge elements and systems – get in, get out, stay out, http://www.fhwa.dot.gov/bridge/prefab/index.htm.

Hewes, J. T., and Priestley, M. J. N. 2002. Seismic design and performance of precast concrete segmental bridge columns, *Report. No. SSRP-2001/25*, University of California, San Diego, San Diego, CA.

Wang, J.-C., Ou, Y.-C., Chang, K.-C., and Lee, G. C. 2008. Large-scale seismic tests of tall concrete bridge columns with precast segmental construction, *Earthquake Engineering and Structural Dynamics*, 37(12), 1449-1465.

Ou, Y.-C., Wang, P.-H., Tsai, M.-S., Chang, K.-C., and Lee, G. C. 2009. Large-scale experimental study of precast segmental unbonded post-tensioned concrete bridge columns for seismic regions. *Journal of Structural Engineering, ASCE,* 10.1061/(ASCE)ST.1943- 541X.0000110.

ASTM. 2004. *ASTM Annual Book,* American Society for Testing and Materials, West Conshohocken, PA.

Schnell, R. E., and Bergmann, M. P. 2007. Improving tomorrow's infrastructure: extending the life of concrete structures with solid stainless steel reinforcing bar, *Proceedings of 3rd NYC Bridge Conference*, New York, NY, U.S.A., Aug. 27-28.

Zatar W. A., and Mutsuyoshi H. 2002. Residual displacements of concrete bridge piers subjected to near field earthquakes, *ACI Structural Journal*, 99(6), 740-749.

Kawashima, K. 2000. Seismic performance of RC bridge piers in Japan: an evaluation after the 1995 Hyogo-ken nanbu earthquake, *Progress in Structural Engineering and Materials*, 2(1), 82-91.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, 4th ED. American Association of State Highway and Transportation Officials (AASHTO), Washington, DC.