Chapter 15

Seismic Behaviour of Post-Tensioned Segmental Bridge Columns with Self-Centring System

Ehsan Nikbakht

Additional information is available at the end of the chapter

http://dx.doi.org/10.5772/65489

Abstract

Prefabricated bridge columns have shown increasing demands over the past few years due to their advantages compared to conventional bridge columns. The reason for this interest is due to their appropriate performance against severe earthquakes (EQs), where they remain functional and repairable with lower amount of cracks and damage. However, there is some uncertainty on application of these kinds of bridge columns in high seismic areas mainly due to lack of knowledge on their behaviour against severe earthquake loading. Therefore, comprehensive design guidelines which consider nonlinearity and energy dissipation and recentring capacity of the post-tensioning (PT) segmental columns are necessary. In this study, continuous steel bars are incorporated as starters through the segments and footing foundation in order to increase the seismic energy absorption of PT segmental columns. The influences of different parameters such as the force level criteria for post-tensioning, steel jacketing and mild steel ratio and column aspect ratio (AR) are important factors which have to be appropriately selected in various design procedures, such as displacement-based design, in order to achieve desirable stiffness, strength, equivalent viscous damping and lateral seismic demand. In this study, the cyclic loading and ground motion excitations were numerically performed in order to evaluate the bridge column seismic demands. Guidelines for performance-based design and displacement-based design are proposed.

Keywords: seismic loading, precast, post-tensioned, bridge columns, design, aspect ratio

1. Introduction

Precast bridge columns have different lateral performances against seismic loading. Due to the uplifting of joints, they exhibit rocking base mechanism avoiding excessive damage in critical areas of columns. In recent years, structures with controlled rocking base mechanism have shown superior performance in comparison with monolithic conventional systems, both for buildings [1–3] and bridge systems [4, 5]. A number of researches have been conducted on unbonded or partially bonded post-tensioned precast components for bridge constructions [6–8]. Several studies have been carried on pseudo-dynamic analysis of single degree of freedom (SDOF) of precast post-tensioned bridge constructions [9] or multi-degree of freedom (MDOF) using nonlinear algorithm [10].

Among post-tensioned precast bridge elements, the focus of this study is to investigate nonlinear-static and pseudo-dynamic behaviour of precast post-tensioned self-centring segmental bridge columns. The definitions and main concepts are presented in the next section.

Precast segmental bridge columns comprise multiple numbers of reinforced concrete (RC) segments connected together by continuous post-tensioning (PT) strands. Continuous post-tensioning strands/bars are placed through the segments using two methods, i.e. bonded and unbonded (**Figure 1**). The unbonded post-tensioning strands are passed through hollow ducts. However, full interaction between strands and surrounded concrete may be achieved in bonded post-tensioning method. Post-tensioning may be implemented on both bars and strands. The benefits of using the strand's post-tensioning in comparison with bar post-tensioning are that strands are able to show higher flexible behaviour; therefore, they can be implemented for any design cross sections. Another advantage of application of strands is that they possess lower stress relaxation both during and after loading as compared to post-tensioned bars.

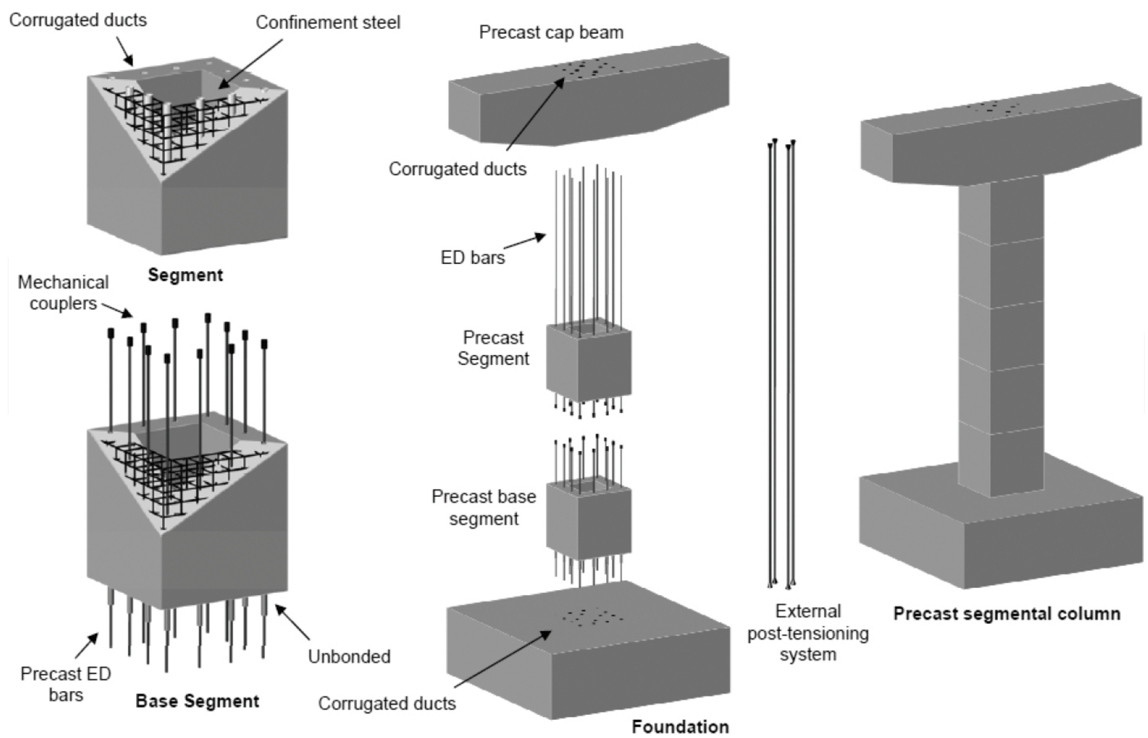

Figure 1. The segmental column tested by Ou et al. [12].

The action of precast segmental bridge columns against seismic loading is different with conventional bridge columns. They act as rocking mechanism. In fact, rocking occurs when segment openings occur, and as a consequence, damages induced are much lower than that of conventional monolithic bridge columns. Minor cracks and damages occurred in this system make them more economical to repair than that of monolithic system after severe earthquake (EQ) event. Various methods for construction of segmental columns have been investigated. Lin and Mo [11] studied segmental column in which the footing foundation and hinge area of the columns were monolithic cast in place. In the aforementioned study, post-tensioning tendon was anchored at the top of the first segment. The disadvantage in this column system is that only limited amount of segments can be assembled on the first segment. Therefore, due to high weight of segments, this type of segmental columns is applicable in shorter bridge columns [12]. Another disadvantage is that there is still hinge area at the base of column, and therefore damage may occur in this area when lateral loading is increased.

Multiple segmented precast columns have been investigated by other researchers such as [12–17]. In the current research presented in this chapter, this type of bridge column is investigated.

Although in recent years some researches have been conducted on precast segmental bridge columns, there is still lack of confidence on application of this kind of columns in high seismic regions due to their high level of peak lateral top displacement and low seismic energy dissipation capacity. The necessity of design guidelines for the segmental bridge columns in high seismic regions is indispensable. In this chapter, fully segmental columns with central strands are investigated. Continuous bonded mild steel starter bars are introduced in the critical area of the columns. Concerted analytical studies have been conducted in order to evaluate seismic demands of precast post-tensioned segmental columns. Design recommendations have been proposed based on analytical investigations in this current study to achieve both high energy dissipation capacity and small residual drift for application at different seismic demands.

2. Nonlinear-static analyses

In this section, reverse cyclic loading analyses are performed in order to investigate the residual displacement, energy dissipation capacity, stiffness and load bearing capacity of precast segmental bridge columns.

2.1. Influence of post-tensioning forces and strand position

In order to investigate the role of strand position on lateral load-carrying capacity, energy dissipation and residual displacement, precast segmental columns with different arrangements of PT strands are analysed and compared with self-centring segmental columns with central PT strands. **Figure 2** shows two segmental columns: one with a 200 mm distance from the centre and one with 140 mm. The total area of the strands in all columns is 2665 mm^2. In order to provide higher energy dissipation capacity, strands are bonded post-tensioning.

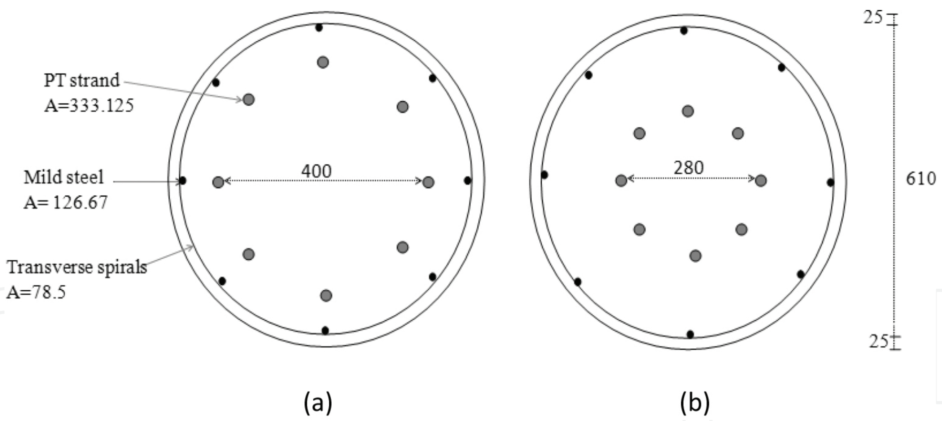

Figure 2. Cross sections of precast segmental columns with different strand positions: (a) BS400 and (b) BS280. Note: All measurements are in mm.

Figure 3 compares the lateral response of columns with a 400 mm distance between strands (BS400), a 280 mm distance (BS280) and the segmental column with central-bonded PT strands (which is explained in the previous section). **Figure 3** indicates that placing strands towards the circumference leads to higher strength than columns with strands at the centre, but there is also higher residual displacement. Although the columns with strands placed near the circumference have the advantage of providing higher energy dissipation capacity, they increase the lateral strength of the columns, which requires designing the other bridge components (e.g. decks) stronger as well. **Table 1** compares the strength and residual displacement of the specimens with different positions of strands. There is another disadvantage: higher strength may cause concrete failure in the hinge area of the columns due to excess concrete failure stress. However, the higher energy dissipation leads to more appropriate lateral performance, with lower lateral peak displacement against severe earthquake loading.

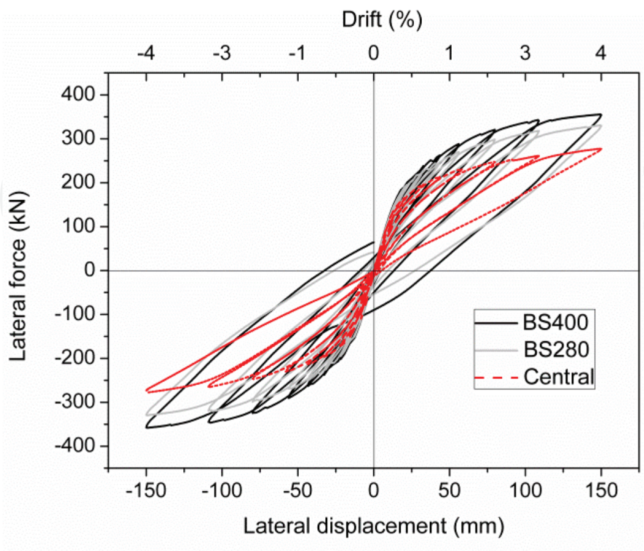

Figure 3. Lateral response of precast segmental columns with different PT strand positions.

Distance from centre (mm)	Strength (kN)	Residual displacement (mm)
0	260	0
140	314	26
200	342	38

Table 1. Strength and residual displacement up to 4.0 % drift level.

In order to investigate the effect of the post-tensioning force level, the segmental column with central PT strands and the BS400 sample are compared under 40 and 70 % initial stress levels in **Figure 4**. As is shown in the figure, the higher initial stress level (70 %) in both columns shows higher initial stiffness. However, the BS400 sample at the 40 % initial stress level shows close lateral strength compared with a 70 % initial stress level over a 4 % drift level. In **Figure 4(b)**, the ultimate lateral strength difference in central post-tensioned columns is more pronounced (261 kN strength under 40 % initial stress level versus 293 kN strength at 70 % initial stress level). The results show that increasing the PT force level has little effect on the residual displacement and energy dissipation capacity of the columns.

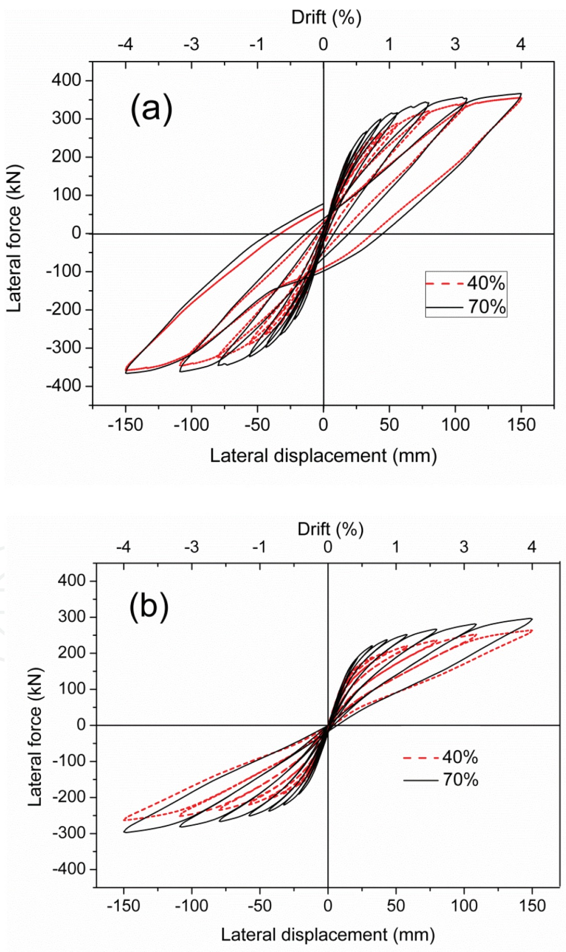

Figure 4. Lateral response of (a) BS400 and (b) segmental columns with central PT strands under 40% and 70 % post-tensioning force levels.

2.2. Influence of steel tube jacketing thickness

In this study, steel tube jacketing thickness is investigated as another parameter. **Figure 5** compares the lateral response of the aforementioned precast segmental columns against central PT strands with a thickness of 3, 6 and 9 mm around the first segment. The figure indicates that increasing the thickness of the steel tube slightly increases the stiffness and strength of the column. Columns with thickness of 3 mm show a strength of 223 versus 275 kN strength for the column with tube jacketing thickness of 9 mm. In **Figure 5(c)**, the segmental columns with tube jacketing thickness of 9 mm fail at 3.6 % drift due to concrete failure below the tube jacketing at the joint of the footing-first segment area. **Table 2** compares behaviour of the segmental columns with different steel tube jacketing up to 4.0 % drift level. In fact, the large thickness of tube jacketing transfers the stress induced in the first segment's hinge area to the junctions of the footing-first segment; as a consequence, concrete in this area fails. The results show that the residual displacement and energy dissipation capacity of the columns with central PT strands are not affected by the thickness of the steel tube jacketing.

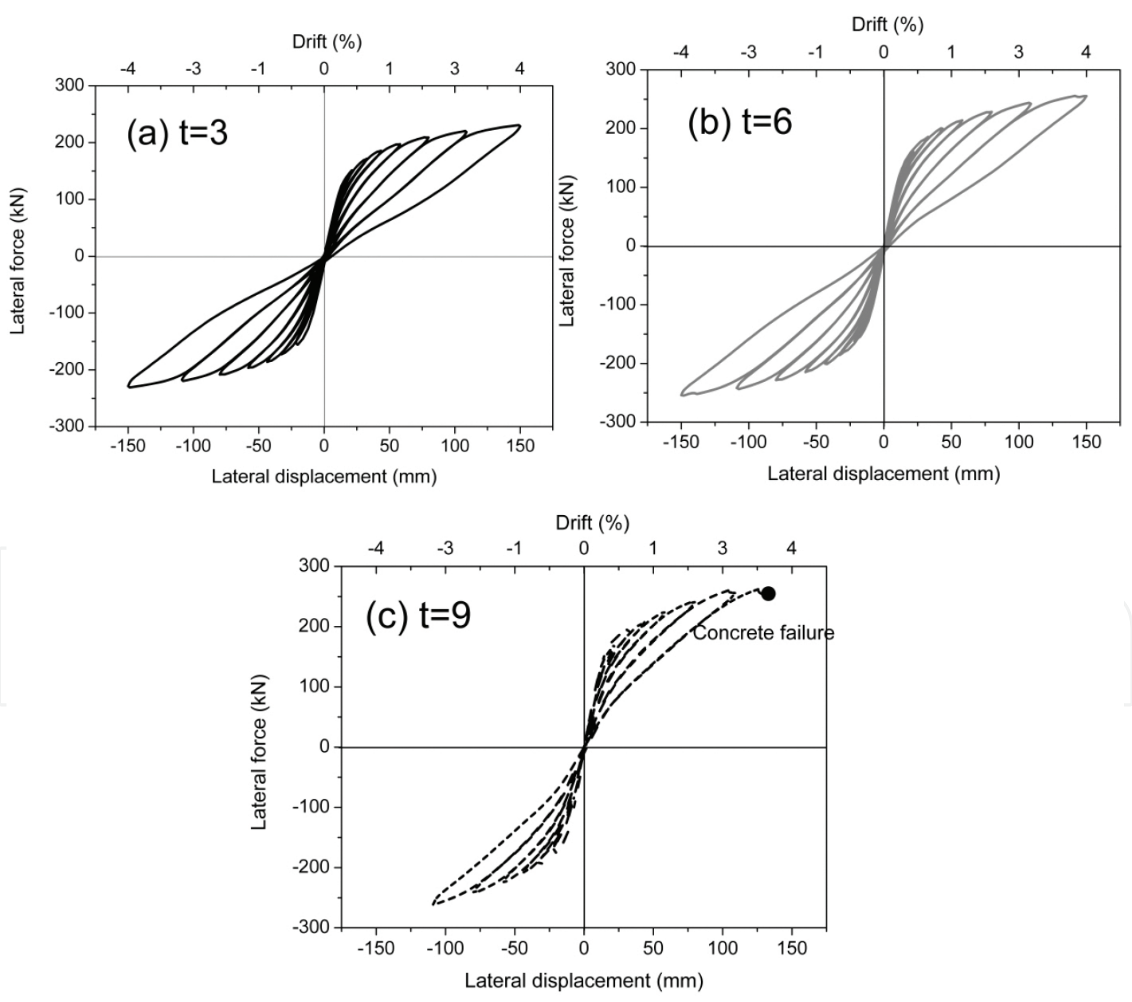

Figure 5. Lateral response of segmental columns with steel tubes with diameters of a) 3 mm, b) 6 mm and c) 9 mm around the first segment.

http://dx.doi.org/10.5772/65489

Thickness of tube jacket (mm)	Stiffness (kN/mm)	Strength (kN)	Failure mode
3	155	212	-
6	172	255	-
9	190	245	Concrete failure at 3.6 % drift

Table 2. Comparison of behaviour of segmental columns with different thickness of steel tube jacketing.

2.3. Influence of aspect ratio (AR) of columns

In this section, segmental columns with aspect ratios (ARs) of 4.5, 6.0, 7.5 and 9.0 are investigated (**Figure 6**). The lateral response of precast segmental columns with different aspect ratios is compared in **Figure 7**. As is shown in the Figure, the lateral strength of the columns decreases when the aspect ratio increases. **Figure 8** compares the equivalent viscous damping of the self-centring segmental columns at aspect ratios of 6.0, 7.5 and 9.0. The figure indicates that equivalent viscous damping of the columns is not affected by aspect ratio, as they show very close response behaviour. Residual displacement in all columns is negligible.

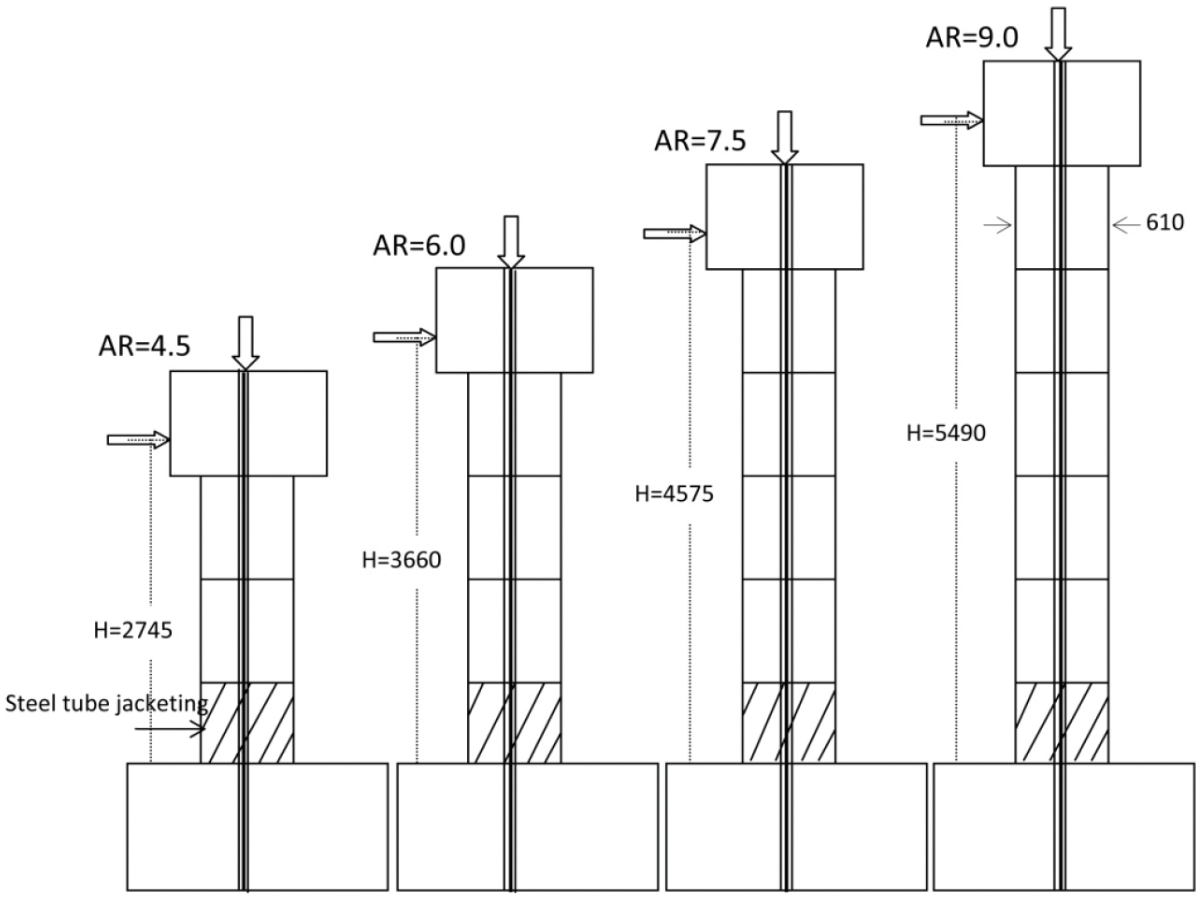

Figure 6. The analysed columns with various aspect ratios of 4.5, 6.0, 7.5 and 9.0.

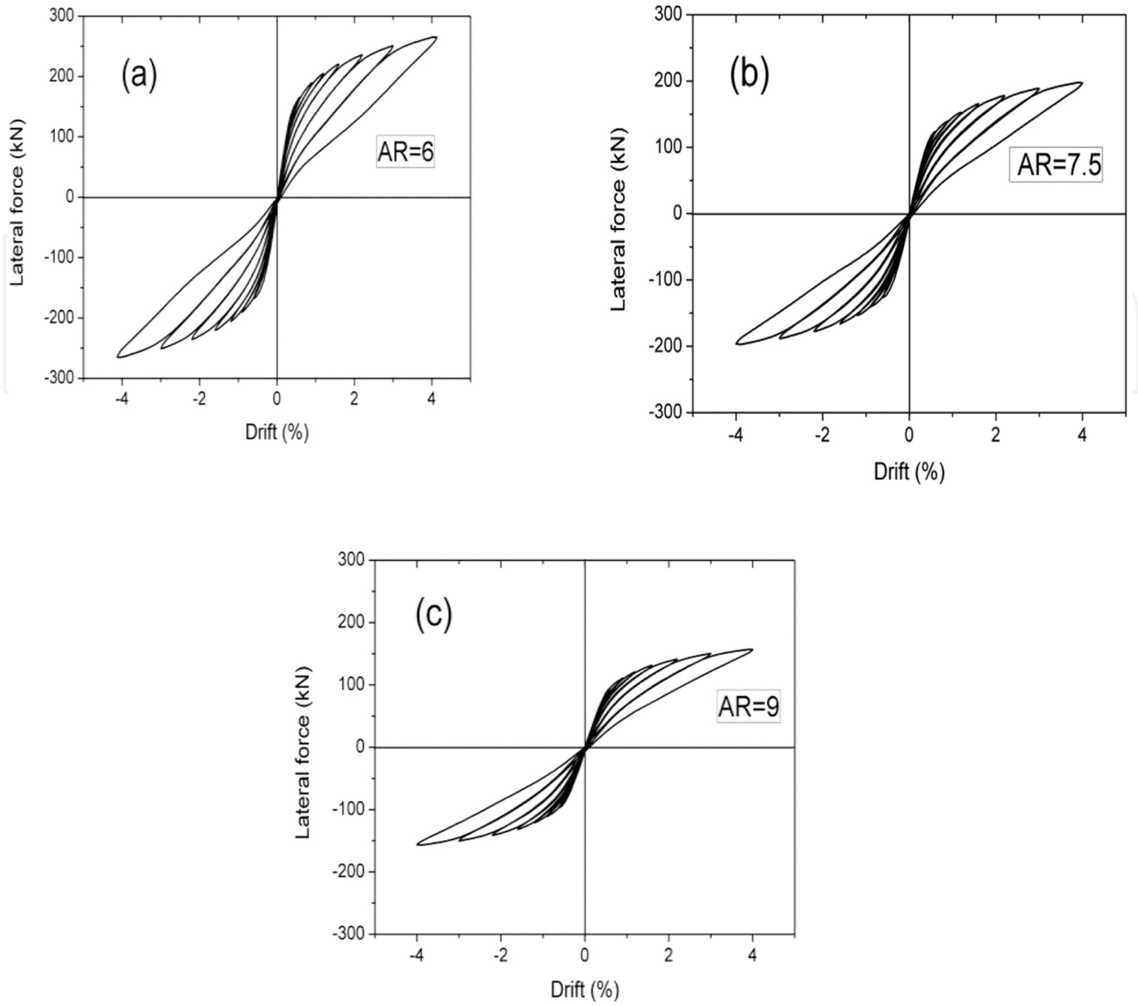

Figure 7. Lateral response of segmental columns with an aspect ratio of (a) 6.0, (b) 7.5 and (c) 9.0.

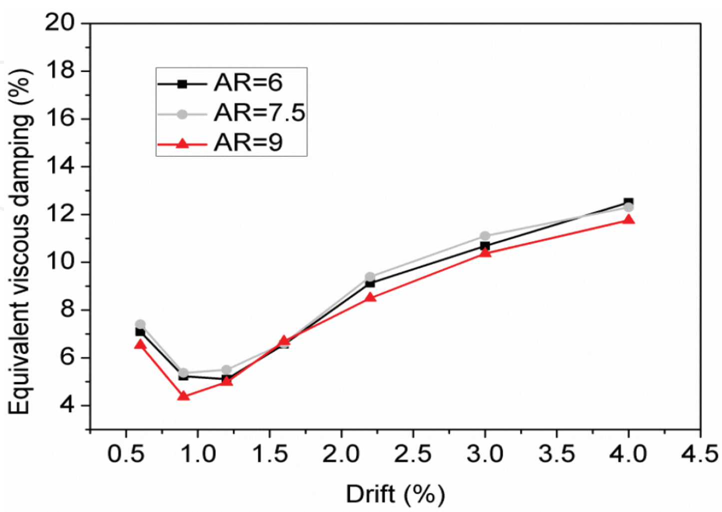

Figure 8. Equivalent viscous damping of self-centring columns at aspect ratios of 6.0, 7.5 and 9.0.

3. Dynamic time-history analyses

In order to examine the dynamic response of precast segmental columns, time-history analysis is conducted. Two earthquake records from Northridge (with 0.41 g) and Loma Prieta (with 0.6 g) are selected for the analyses. The applied earthquake time histories are shown in **Figure 9**. The characteristics of ground motions are presented in **Table 3**.

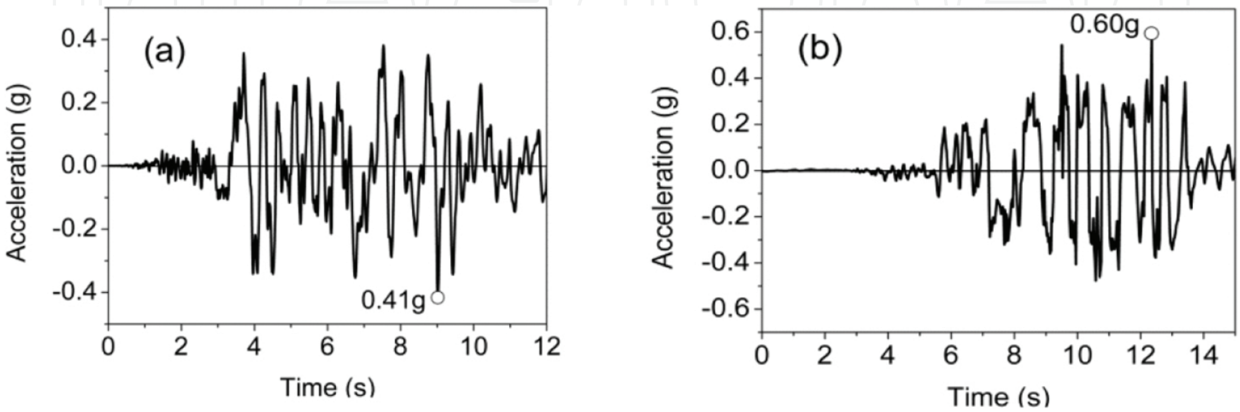

Figure 9. Earthquake records applied in the analyses: (a) Northridge and (b) Loma Prieta.

No.	Event	Year	Station	M[a]	R[b](km)	peak ground acceleration (g)	peak ground acceleration/peak ground velocity
1	Northridge	1994	Castaic—old Ridge Route	6.69	20.7	0.41	0.85
2	Loma Prieta	1989	BRAN	6.93	10.7	0.6	1.14

Source: Pacific Earthquake Engineering Research center strong motion database. [a]Moment magnitude; [b]closest distance to fault rupture

Table 3. Applied earthquake ground motion records.

As discussed earlier, the self-centring segmental columns possess low energy dissipation capacity due to the restoration capability of strands during the unloading stages. It is predominantly the segments' opening and low energy dissipation which leads to the high level of peak lateral displacement. In this section, the effect of PT force levels and the continuous mild steel ratio are investigated with regard to lateral peak displacement demand of segmental columns with central PT strands.

For the analysis, a column with four segments, a height of 3.325 m and an aspect ratio of 6.0 is used. A seismic mass of 1000 kN equivalent to the axial superstructure dead load and one-third of the column's mass is applied. According to the recommendation of AASHTO [18], a 5.0 % damping ratio is applied in the analyses.

3.1. Results and discussion

In **Figure 10**, the lateral peak displacement of the segmental column at 40 and 70 % initial stress levels is compared under the Northridge earthquake record. The sample with a 70 % initial stress level is expected to show close or lower lateral peak displacement due to higher stiffness and strength. **Figure 10(a)** compares the lateral responses of the column with 0.41 % mild steel ratio at 40 and 70 % initial stress. As is shown, this sample at both initial stress levels shows close lateral peak displacements, i.e. the peak lateral top displacement at both initial stress levels is around 113.0 mm. However, in **Figure 10(b)**, when the continuous mild steel ratio is zero, higher peak lateral displacement is shown at the 70 % initial stress level, i.e. 145.9 mm peak lateral displacement versus 118.5 mm at the 40 % initial stress level, which is undesirable. In fact, when the initial stress level is 70 %, the higher amount of stiffness and strength, alongside an inadequate amount of energy dissipation in the column with 0 % mild steel ratio, leads to a concrete crush in the hinge area. This causes considerable stiffness loss and consequently higher lateral peak displacement.

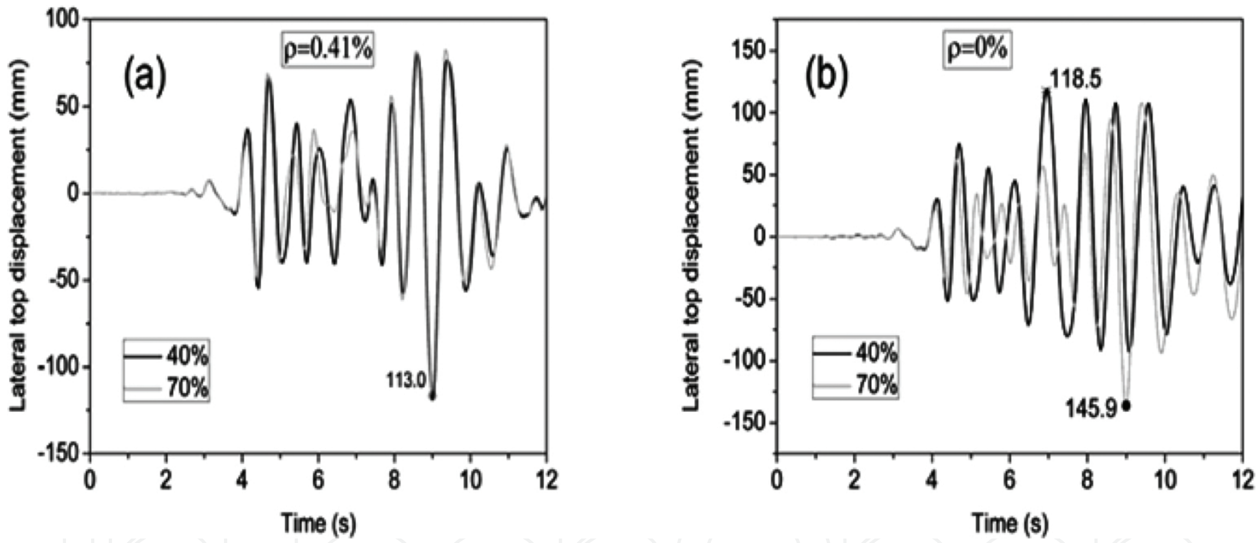

Figure 10. Lateral seismic response of segmental columns at 40% and 70 % initial stress levels under Northridge EQ with mild steel ratios of (a) 0.41 % and (b) 0.0 %.

Figure 11 compares the lateral peak displacement of the segmental columns with mild steel ratios of 0.17 and 0.52 %, subjected to both earthquake ground motions. In **Figure 11(a)**, under the Loma Prieta earthquake record, the segmental column with a 0.52 % steel ratio shows 18 % lower lateral peak displacement than the columns with a 0.17 % mild steel ratio, i.e. 190.2 versus 236.0 mm lateral peak displacement of the column with a 0.17 % steel ratio. Similar results can be seen in **Figure 11(b)** where the column with a 0.52 % mild steel ratio shows 23 % lower lateral peak displacement under the Northridge earthquake record.

http://dx.doi.org/10.5772/65489

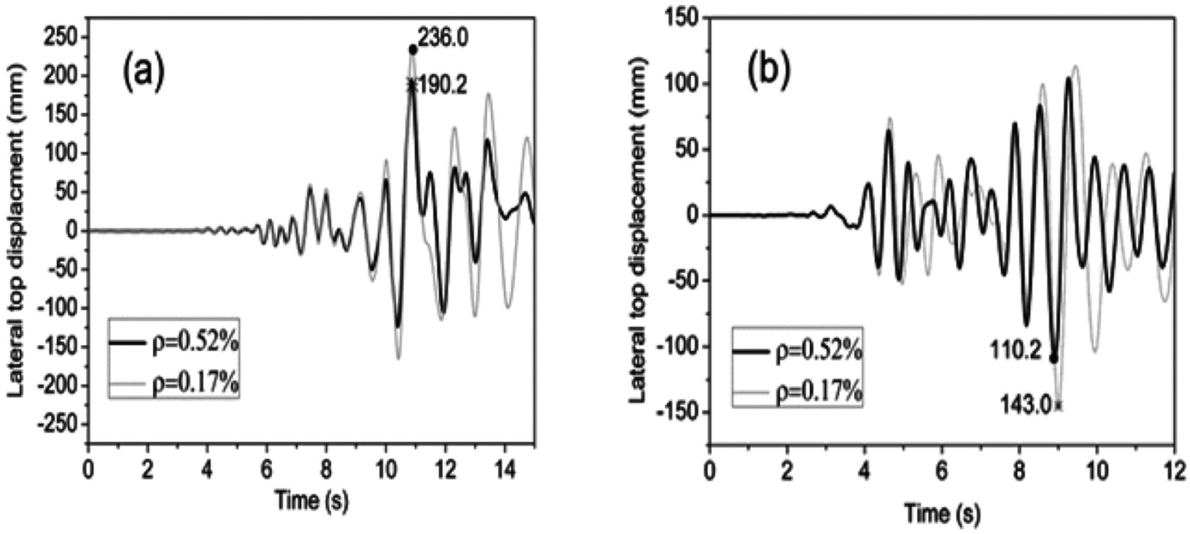

Figure 11. Effect of mild steel ratio on lateral seismic demand of segmental columns subjected to (a) Loma Prieta and (b) Northridge EQ records.

4. Conclusions

The following conclusions can be made based on numerical investigation conducted in this study:

1. The segmental columns with strands near the circumference of the cross section showed higher stiffness and strength than the columns with central strands but also larger residual displacement. In addition, the column with central strands at a 70 % prestressing level showed higher strength than a 40 % initial stress level, while the columns with strands around the cross section exhibited similar strength at both 40 and 70 % prestressing levels.

2. PT segmental columns with prestressing force level of 40 % of tensile stress of strands exhibited lower lateral peak displacement than the other samples.

3. Increasing the thickness of steel jacket around the first segment corresponded with an increase to the stiffness and strength of the columns. However, it had a little effect on residual displacement and energy dissipation capacity of the self-centring columns. It was shown that large thickness of steel tube jacket might cause excessive stresses in critical areas at large drifts, e.g. the column with a steel tube jacket thickness of 9 mm failed at 3.6 % drift due to concrete failure below the steel jacketing areas.

4. The analyses of the segmental columns with aspect ratios of 4.5, 6, 7.5 and 9 indicated that columns with a higher aspect ratio show lower amount of strength and energy dissipation. However, it was shown that aspect ratio has little effect on equivalent viscous damping of the columns.

5. Introducing continuous longitudinal mild steel ratio in higher aspect ratio columns caused lower residual drift and equivalent viscous damping than those with lower aspect ratio. It was

shown that there is a greater capacity for increasing the amount of mild steel ratio in higher aspect ratio columns until their residual drift exceeds the limitation of 1 %. Specifically, the maximum allowed mild steel ratio for the segmental column with aspect ratio of 9.0 was 60 and 23 % larger than that of the columns with aspect ratios of 6.0 and 7.5, respectively.

6. Time-history analyses indicated that the columns with inadequate amount of mild steel ratio and higher level of post-tensioning exhibited greater lateral peak displacement response than those with higher mild steel ratio and lower post-tensioning level. The column with a 0.52 % mild steel ratio showed 19 % lower lateral peak displacement than that with a 0.17 % mild steel ratio under Loma Prieta earthquake record and displayed 23 % lower top displacement under Northridge record.

Author details

Ehsan Nikbakht

Address all correspondence to: ehsan.nikbakht@petronas.com.my

Department of Civil and Environmental Engineering, Universiti Teknologi Petronas, Bandar Seri Iskandar, Malaysia

References

[1] Priestley M.J.N., Sritharan, S., Conley, J.R. & Pampanin S. Preliminary results and conclusions from the PRESS, five-story precast concrete test building. PCI Journal. 1999; 44(6):42-67.

[2] Kurama, Y., Sause, R., Pessiki, S. & Lu, L. Seismic response evaluation of unbonded post-tensioned precast walls. ACI Structural Journal. 2002; 99(5):641-651.

[3] Pampanin, S., Priestley, M. J. N., & Sritharan, S. Analytical modeling of the seismic behavior of precast concrete frames designed with ductile connections. Journal of Earthquake Engineering. 2001; 5(3):329-367.

[4] Mander, J.B. & Cheng, C.T. Seismic resistance of bridge piers based on damage avoidance design. Technical Report NCEER-97-0014, State University of New York, Buffalo, NY, 1997.

[5] Palermo, A., Pampanin, S., & Calvi, G.M. Concept and development of hybrid solutions for seismic resistant bridge systems, Journal of Earthquake Engineering. 2005; 9(6): 899-921.

[6] Kawashima K. Seismic design of concrete bridges. 1st fib Congress, Osaka, Japan, 2002.

[7] Ikeda, S., Hirose, S., Yamaguchi, T. & Nonaka, S. Seismic performance of concrete piers prestressed in the critical sections. Proc., 1st fib Congress, Osaka, Japan, 2002, 207-214.

[8] Marriott, D., Pampanin, S., & Palermo, A. Quasi-static and pseudo-dynamic testing of unbonded post-tensioned rocking bridge piers with external replaceable dissipaters, Earthquake Engineering & Structural Dynamics. 2008; 38:331-354.

[9] Dhakal R.P., Mander JB & Mashiko N. Identification of critical ground motions for seismic performance assessment of structures. Earthquake Engineering and Structural Dynamics. 2006; 35(8):989-1008.

[10] Pinto, A., Pegon, P., Mangonette, G. & Tsionis G. Pseudo-dynamic testing of bridges using non-linear substructuring. Earthquake Engineering and Structural Dynamics. 2004; 33:1125-1146.

[11] Lin, J.C. & Mo, Y.L. The shear transfer behavior of precast prestressed hollow rectangular bridge columns. M. S. Thesis, Dept of Civil Engineering, National Cheng Kung University, Tainan, 2002.

[12] Ou, Y.-C., Chiewanichakorn, M., Aref, A.J., & Lee, G.C. Seismic performance of segmental precast unbonded post-tensioned concrete bridge columns, Journal of Structural Engineering. 2007; 133(11):1636-1674.

[13] Motaref, S., Saiidi, M.S. & Sanders, D. Shake table studies of energy-dissipating segmental bridge columns. Journal of Bridge Engineering 2014; 19:186-199.

[14] Sideris, P., Aref, A.J. & Filiatrault, A. Quasi-static cyclic testing of a large-scale hybrid sliding-rocking segmental column with slip-dominant joints. Journal of Bridge Engineering. 2014; 10, 04014036-1-11.

[15] Nikbakht, E., Rashid, K., Siti, A.O., & Hejazi, F. A numerical study on seismic response of self-centering precast segmental columns at different post-tensioning forces. Latin American Journal of Solids and Structures. 2014; 5:864-883.

[16] Nikbakht, E., Rashid, K., Mohseni, I, & Hejazi, F. Evaluating seismic demands for segmental columns with low energy dissipation capacity. Earthquakes and Structures. 2015; 8:1277-1297.

[17] Nikbakht, E., Rashid, K., Hejazi, F., & Osman, S. A. Application of shape memory alloy bars in self-centering precast segmental columns as seismic resistance. Structure and Infrastructure Engineering. 2015; 11:297-309

[18] AASHTO. Standard Specifications for Buildings (16th ed.). Author: Washington, DC, 1996.