Seismic performance of precast segmental columns: shake table tests

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Abstract

Precast segmental columns are more and more widely used in the construction industry due to its obvious advantages such as the fast construction speed, improved construction quality and reduced environmental impact. To understand the seismic performances of segmental columns, extensive studies have been carried out recently. However, these studies mainly focused on the quasi-static cyclic tests to obtain the force-displacement relationship of the column by using actuators. Moreover, most previous studies only considered the loading in one direction. In reality, earthquake loading has three components, which may result in different responses compared to the uniaxial input. This paper carries out experimental studies on the seismic performances of precast segmental columns by using the newly commissioned shaking table system in Curtin University. Earthquake loadings in the two horizontal directions were used as inputs and the dynamic responses of the column under the biaxial earthquake excitations with different peak ground accelerations (PGAs) are examined. For comparison, the seismic responses of the conventional monolithic column were also reported.

Keywords: segmental column; seismic performance; shake table tests; biaxial earthquake loadings.

1. INTRODUCTION

In recent years, precast segmental column, which uses post-tensioned tendons to clamp the prefabricated segments together, has been more and more widely used in the construction industry due to its obvious advantages such as better quality control, reduced site construction activities, etc. However, its applications are still limited due to the insufficient knowledge about its performance under earthquake loading (Billington and Yoon 2004, ElGawady et al. 2010, Li et al. 2017a).

Some studies have been carried out recently to investigate the seismic performances of precast segmental columns (Billington and Yoon 2004, Cai et al. 2018, Cha et al. 2018, Chou and Chen 2006, ElGawady et al. 2010, Guerrini et al. 2015, Hewes and Priestley 2002, Li et al. 2018, Li et al. 2017b, Ou et al. 2009, Wang et al. 2008, White and Palermo 2016). However, most of these studies focused on the quasi-static cyclic test to obtain the force-displacement relationship of the column. Only very limited number of shake table tests have also been carried out to evaluate its dynamic performances (Motaref et al. 2013, Moustafa and ElGawady 2018, Yamashita and Sanders 2009). However, only the uniaxial earthquake loading was considered in these studies. In reality, earthquake loading has three components, using one directional input may not be able to reflect the real seismic response of the column. This paper carries out shake table tests on the seismic responses of a precast segmental column subjected to the biaxial horizontal earthquake loadings by using the newly commissioned four-shake table system in the Structural Dynamics Lab at Curtin University. For comparison, a conventional monolithic column was also experimentally investigated.

2. COLUMN DESIGN

Two columns, i.e. one segmental column and one monolithic column, were designed and fabricated in this study. The column was scaled down from a real bridge pier prototype. The prototype column has a diameter of 1.22 m and a height of 7.32 m. Considering the capacity of the shake table, a scale factor of 1/12 was selected. Therefore, a diameter of 100 mm and height of 600 mm were chosen for the model column. Fig. 1 shows the design of the two columns. For the monolithic column (Fig. 1(a)), the footing, column and the cap were cast together as a whole part. Four steel bars with a diameter of 6 mm were used as the longitudinal reinforcement. The diameter and spacing of the stirrups were 4 mm and 35 mm, respectively. In contrast, for the segmental column, the column was divided into three segments. As shown in Fig. 1 (b), the segments, footing and the cap were cast separately and then installed in the lab. The steel reinforcement were the same as the monolithic column, the only difference was that the longitudinal bars were not continuous across the joints between the segments. A post-tensioned strand with a cross sectional area of 54.7 mm^2 was used to clamp all the segments together. The tendon was anchored at the footing and the top surface of the cap. The post-tensioned force applied to the column was 24.8 kN, which was approximately 8.7% of the axial load capacity of the column, i.e. $f_c' A_g$, where f_c' is the concrete compressive strength (38 MPa as measured) and A_g is the gross section area of the column. Two pieces of mass blocks were fixed on the column top to mimic the weight of the superstructures. The dimensions of the block were 1000 × 1000 × 150 mm (length × width × height) and the weight of each slab was 375 kg. The total mass applied on the column, including the mass blocks and the cap was 844 kg. Table 1 shows the properties of the materials used in the test, where ρ is density, E Young's modulus, f_c' concrete compressive strength, f_t tensile strength of the concrete and f_y the characteristic yield strength of the steel rebars or prestress tendon.

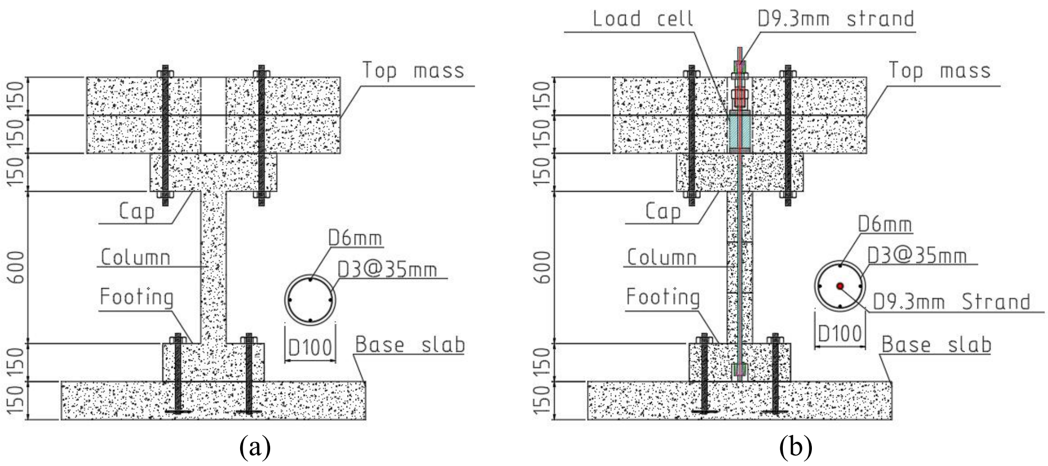


Fig. 1. Design details of the two columns: (a) monolithic column, (b) segmental column.

Table 1. Material properties

Material	ρ (kg/m^3)	E (GPa)	f_c' (MPa)	f_t or f_y (MPa)
Concrete	2400	30	38	5
Longitudinal rebar	7800	200	-	500
Stirrup	7800	200	-	300
Prestress tendon	7850	195	-	1860

3. TEST SETUP

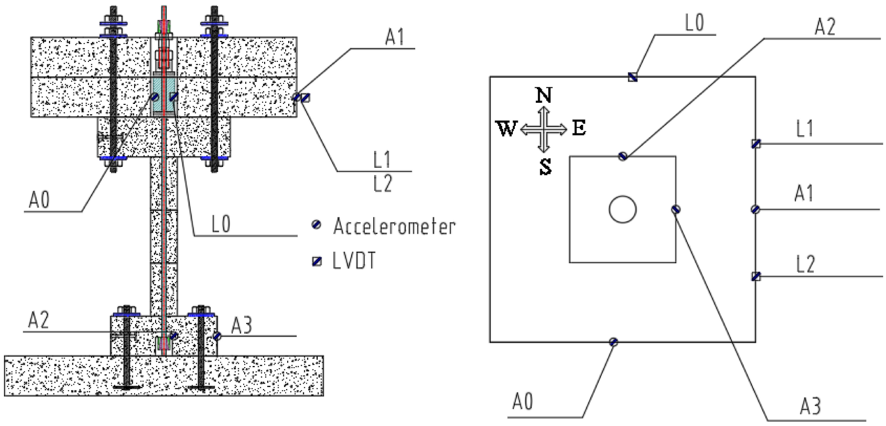
Each table has a platform with the dimensions of 1000 × 1000 mm (length × width). The maximum payload, frequency range and peak stroke of each table are 8 kN, 0.1-50 Hz, and ±150 mm, respectively. With the column as designed in Fig. 1, one shake table was not powerful enough for the test. The four shake tables were therefore used together in the test and a base slab with a dimension of 1500 × 1500 × 150 mm, which was connected to the four tables was used as the base to support the test column specimen. Fig. 2(a) shows the arrangement of the shake table and Fig. 2(b) shows the setup of the specimen. With such design, the synchronized movements of the four shake tables are very critical. They were carefully controlled during the tests and the recorded data showed very good synchronization between them.

Sensors including LVDTs and accelerometers were used to capture the responses of the column. Fig. 5 shows the layout of the sensors. Two accelerometers (A0 and A1) were placed on the mass block to measure the accelerations in the North-South and East-West directions and another two accelerometers (A2 and A3) were installed on the footing to measure the input accelerations. Three LVDTs were used to measure the displacements of the mass block. As shown in Fig. 5, LVDT L0 was installed in the N-S direction and LVDTs L1 and L2 were installed in the E-W direction. All the data were recorded by the HBM data acquisition system with a sampling frequency of 200 Hz.

The columns were subjected to the two directional earthquake loadings that were recorded at the Niland Fire Station from the 1979 Imperial Valley Earthquake. The peak ground accelerations (PGA) of the original data were 0.108g and 0.068g in the N-S and E-W directions respectively. In the tests, the original earthquake loadings were scaled with the maximum PGA varying from 0.2g to the collapse of the column with an interval of 0.1g. To account for the scale factor of the test column, the time duration of the input accelerations was compressed by $\sqrt{12} = 3.46$ times. Fig. 4 shows one pair of the earthquake loadings with a maximum PGA of 0.2g.

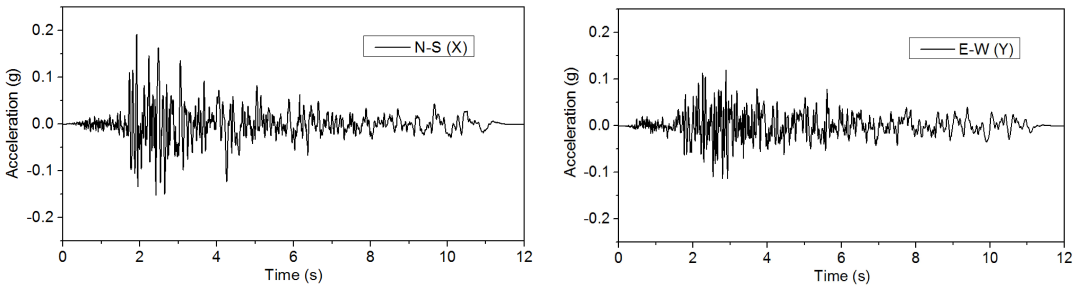
(a) (b)

Fig. 2. (a) Shake table arrangement, (b) test setup



(a) (b)

Fig. 3. Arrangement of sensors: (a) front view; (b) top view



(a) (b)

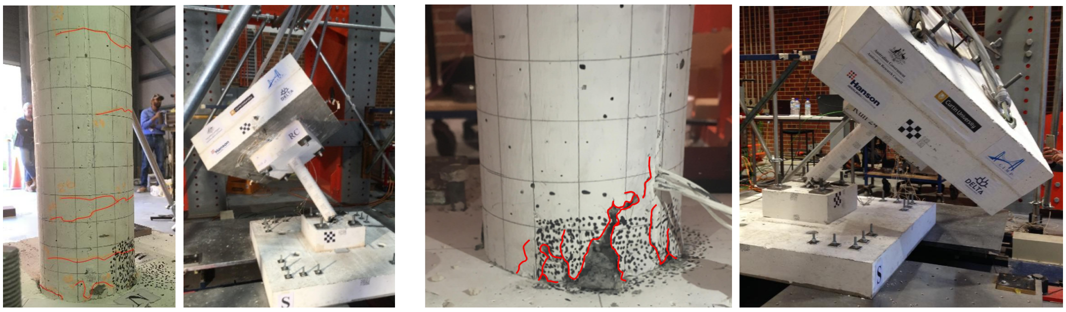
Fig. 4. One pair of inputs (maximum PGA=0.2g): (a) N-S direction; (b) E-W direction

4. TEST RESULTS

Fig. 5 shows the damages of the two columns. For the monolithic column, as shown in Fig. 5(a), the concrete cracks and cover concrete spalling were observed during the test. The horizontal flexural cracks distributed from the bottom of the column and developed to about half height of the column. Fig. 5(b) shows the damage of the precast segmental column. It can be observed that the damage of the precast segmental column was the concrete compressive damage near the joint between the footing and the bottom segment. No tensile crack was observed during the tests. This is because the segments could rock against each other and opening could be formed at the joints instead of forming tensile cracks as observed in the monolithic column.

However, since the bottom segment rocked against the footing, the toes of the segment experienced large compressive stress, which caused concrete crush damage as observed in Fig. 5(b).

The monolithic column collapsed at the maximum PGA of 1.0g, and the segmental column collapsed at the maximum PGA of 0.8g. The segmental column collapsed earlier than the monolithic column. This could be attributed to the large axial force from the post-tensioned tendon. On one hand, the tendon could pull the column to its initial position, so that the residual displacement of the segmental column could be minimized (refer to Fig. 7(b)), on the other hand, the tendon increased the axial stress of the column and the bottom segment experienced excessive compressive stress resulting from the rocking of the column. Concrete crushing damages were therefore formed and developed in the segment, which finally caused the collapse of the column. Therefore, the post-tensioning force should be carefully chosen in the design to balance the pros and cons, and the bottom segments might need to be confined (e.g. by FRP or steel tube) to minimize the concrete crushing and spalling damages.



(a) (b)

Fig. 5. Damage pattern of the two columns: a) monolithic column; b) segmental column

Fig. 6 shows the variations of the first vibration period of the both columns during the tests. It can be seen that when the PGA was relatively small (from 0.2g to 0.6g), the periods of the segmental column S1 were almost a constant with an increment of 8.2% only. For the monolithic column, the period, however, increased from 0.42s to 0.52s, with an increment of 21.6%. These results indicated that for the monolithic column experienced more intensive damages than the segmental column owing to concrete tensile cracks and steel bar yielding, which, do not occur to the segmental column since the joints could open under tensile force. The primary damage to the segmental column is crushing damage to concrete segments and footing as observed above, which was not prominent under these excitation levels. When the PGA was larger, both the vibration periods of the segmental and monolithic columns increased due to the damage developed in the columns. However, the increment was more obvious in the monolithic column as shown in Fig. 6. This might be regarded as an advantage of segmental column, i.e. it is less vulnerable to the repeated seismic excitations that may be experienced during its whole lifecycle.

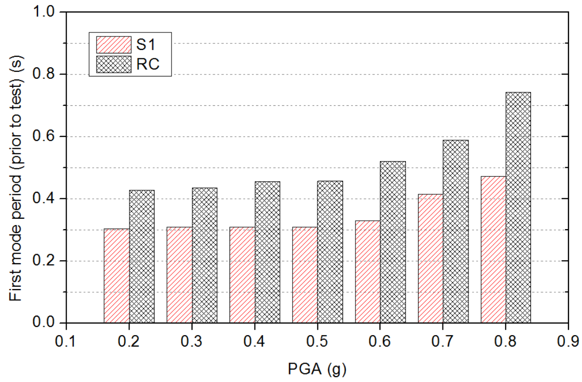
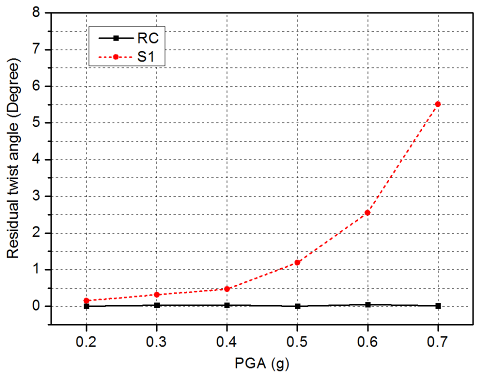


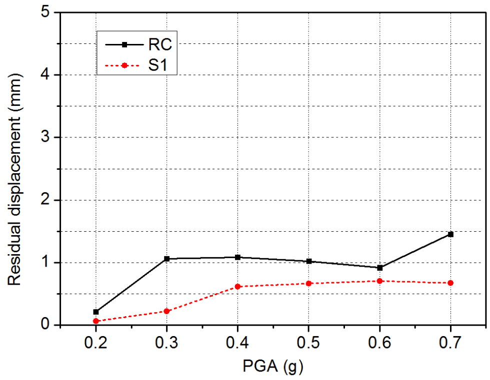
Fig. 6. The first vibration periods of the segmental and monolithic columns after each test

During the tests, it was observed that the monolithic column experienced almost no twisting response although the inputs in the two horizontal directions are different, while the precast segmental column experienced severe twisting. Fig. 7(a) shows the residual twisting angles of the two columns. It can be seen that the twisting angle was almost zero during the tests for the monolithic column, while for the segmental column, it increased from zero to around 5.5 degrees after the maximum PGA reached 0.7g. This is because the only resistance to shear and torsional moment at the joints of segmental column was the friction, which was not enough to resist the torsional moment resulting from the biaxial earthquake loadings. For the monolithic column, since the column was casted as a whole, no such problem existed and the twisting angle was thus small. It should be noted that most previous studies focused on the uniaxial cyclic performances of segmental columns, and it was generally believed that the friction between the joints was sufficient to resist the shear force. However, the present experimental results showed that the friction force between the joints was not enough to resist the torsional moment resulted from the biaxial seismic loadings. Therefore, shear stress resistant links (e.g. concrete shear keys, continuous steel bars) between the joints may need to be provided to increase the torsional resistance of the column.

As mentioned above, LVDTs were used to measure the displacements, and the measured data included the influence from twisting. Fig. 8 shows the schematic view of the column with a twisting angle. The solid lines are the original position of the column and the dash lines represent the column without any lateral residual displacement but with a twisting angle. The red short lines ME in the E-W direction and RR' in the N-S direction are the lateral displacements resulting from twisting. Fig. 7(b) shows the residual displacements of the both columns in the E-W direction after the influence of twisting was removed. It can be found that the residual displacements were small for the both columns. The reason is that the both columns experienced insignificant concrete damage during the tests, and no obvious plastic hinge was formed in the monolithic column as shown in Fig. 5(a). It also can be seen that the residual displacements of the segmental column were smaller than those of the monolithic column due to the fact that the post-tensioned tendon could pull the segmental column back to the original position.



(a)



(b)

Fig. 7. Residual responses of the two columns: (a) residual twisting angles, (b) residual displacements in the E-W direction

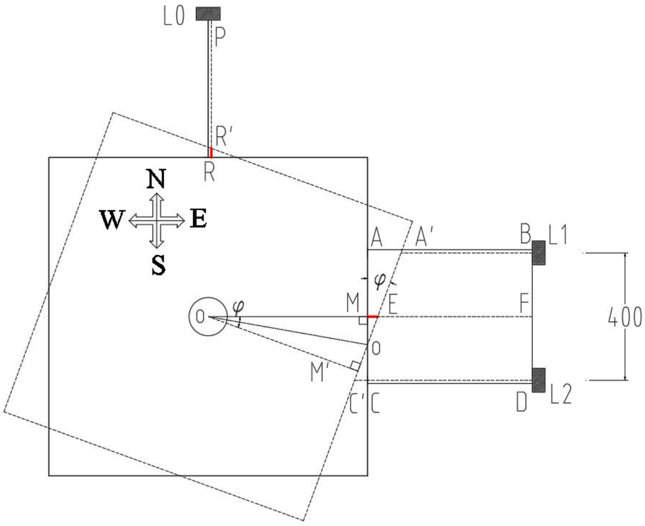


Fig. 8. Schematic view of the column with a twisting angle

5. CONCLUSIONS

In this study, shake table tests were carried out to investigate the dynamic performances of the precast segmental column. Biaxial earthquake loadings with different intensities were used as inputs. For comparison, a conventional monolithic column was also tested as a reference column. Testing results showed that the damage in the monolithic column were widely distributed along the column owing to tensile cracks to the concrete and yielding of the reinforcement bars, could develop to half-height of the column, while for the segmental column, the damage mainly concentrated at the toe of the bottom segment associate to the crushing damage to concrete segment and footing. These damages led to the vibration periods of both columns decreased with the increment of earthquake intensity, and the decrement was more obvious in the monolithic column. Moreover, severe twisting was observed in the segmental column, which indicated the resistance between the joints was not sufficient to resist the torsional moment resulted from the biaxial earthquake loadings. Shear links between the joints might be necessary to increase the torsional resistance of the precast segmental column.

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