Influence of column types on the seismic responses of bridge structures

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Abstract

Segmental columns are more and more widely used in bridge structures recently. Previous studies on the seismic responses of segmental columns mainly focused on the column itself, the investigation on the seismic response of a whole bridge system supported by segmental columns is rare. This paper carries out numerical studies on the seismic responses of a bridge structure supported by segmental columns or traditional monolithic columns. The influence of column types on the bridge seismic responses is discussed and in particular the pounding responses are highlighted.

Keywords: bridge structures; segmental column; monolithic column; seismic response; pounding

1. INTRODUCTION

Compared to the conventional cast-in-place monolithic columns, precast segmental columns as a new type of substructure have many obvious advantages, e.g., better prefabrication quality, reduced environmental impact and traffic disruption. Bridge structures supported by segmental columns are therefore more and more widely constructed recently. However, it should be noted that their applications are mainly limited to the regions with low seismic intensity due to the lack of understanding on its seismic performances.

Some research works (e.g. Ou et al. 2010; Li et al. 2017) have been carried out recently to understand the seismic behaviours of segmental columns. Previous studies revealed that compared to the traditional monolithic columns, there are two main characteristics for the segmental columns. Firstly, the pre-stressed tendons in the segmental columns can provide excellent self-centring capability to the column, which makes the residual displacement of segmental column much smaller than the traditional monolithic column. Secondly, less energy is dissipated by the segmental column because the column is not continuous but segment by segment, the yielding of rebars which dissipates energy is therefore less likely to occur. Many research works were then performed to increase the energy dissipation capacity of segmental columns (e.g. ElGawady and Sha'lan 2010).

Compared to the extensive studies on the seismic performances of segmental column itself, the investigations on the seismic responses of a whole bridge structure supported by segmental columns are rare and no study investigates the influence of column types on the bridge seismic responses. This paper carries out numerical studies on the seismic responses of a bridge structure supported by segmental columns or traditional monolithic columns. Numerical results show that the column types can significantly influence the seismic responses of bridge structures.

2. BRIDGE MODEL

A typical five-span continuous bridge extensively investigated by other researchers (e.g. Megally et al. 2002) is adopted in the present study as the reference bridge with minor modifications on the span length and pier height. Fig. 1 shows the elevation view of the bridge and Fig. 2(a) shows the cross section of the box-girder. It can be seen that the bridge consists of 5 spans, the length of the three middle spans is 30 m and the two side spans is 20 m each. The height of the columns is 10 m. Three expansion joints exist in the bridge, with one located at the middle of the bridge and the other two at the left and right ends. The size of the expansion joints is 0.1 m.

Two types of columns, i.e. the segmental columns and monolithic columns, are considered in the present study. For the segmental columns, the specimen experimentally investigated by Wang et al. (2008) is directly used and Fig. 2(b) shows the details of the segmental column. As shown, six 7T D15 (15mm in diameter) pre-stressed tendons are installed to provide the pre-stressed force and self-centring capability to the column. D22 longitudinal mild steel bars are used in each segment to position the transverse reinforcements (the stirrups, which are not shown in the cross section). D36 energy dissipation (ED) bars are extended continuously in some segments (S1 to S5) of the column.

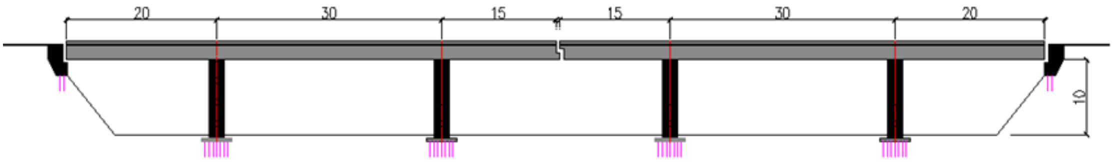
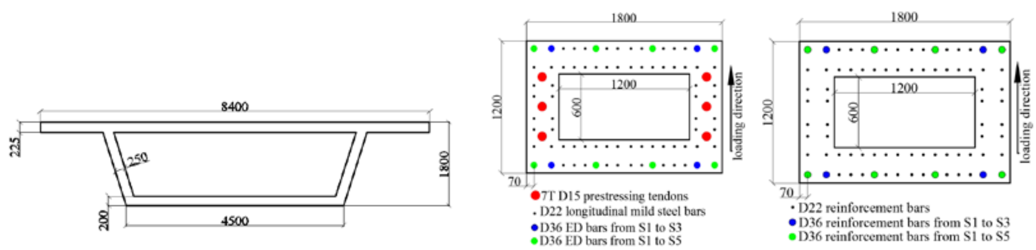


Fig. 1. Elevation and cross section of the bridge model (m)



(a) (b) (c)

Fig. 2. Details of the bridges, (a) cross-section of box-girder; (b) details of the precast segmental column and (c) details of the monolithic column section (mm)

Fig. 2(c) shows the cross section of the monolithic column. For a fair comparison, the longitudinal reinforcement ratio in the segmental column and monolithic column are designed almost the same. Particularly, the same number of longitudinal mild steel bars is used in the monolithic column but they are extended along the whole height of the column. No pre-stressed tendons are designed in the monolithic column. For easy reference, the parameters of the girder and columns are summarized in Table 1.

Table 1 Cross-sectional properties and reinforcing steel ratios of structural components

Structural component	Cross sectional area, A (m^2)	Moment of inertia, I (m^4)	Tendon ratio (%)	ED bar ratio (%)	Longitudinal steel bar ratio (%)
Girders	3	1.74	/	/	/
Segmental columns	1.44	0.24	0.41	S1-S3:0.85 S3-S5:0.57	1.6 (in segment only)
Monolithic columns	1.44	0.24	/	S1-S3:0.85 S3-S5:0.57	1.6 (continuous)

3. NUMERICAL MODELS

The finite element code OpenSEES is adopted to develop the numerical models of the bridge. Bi et al. (2017) systematically presented the numerical modelling and validation of the bridge structures supported by segmental columns or monolithic columns. For completeness of the paper, the numerical models are briefly introduced in this section.

The segmental column is represented by a lumped-mass model together with a zero-length element with “Pinching4” material to capture the global hysteretic behaviours of the column. The accuracy of the simplified model was validated by a previous experimental study (Wang et al. 2008) in Bi et al. (2017). The monolithic column is modelled by the fibre-based model. The unconfined concrete, confined concrete and steel rebars are represented by different fibres with different material properties. This model was validated by another experimental study (Taylor et al. 1997) in Bi et al. (2017).

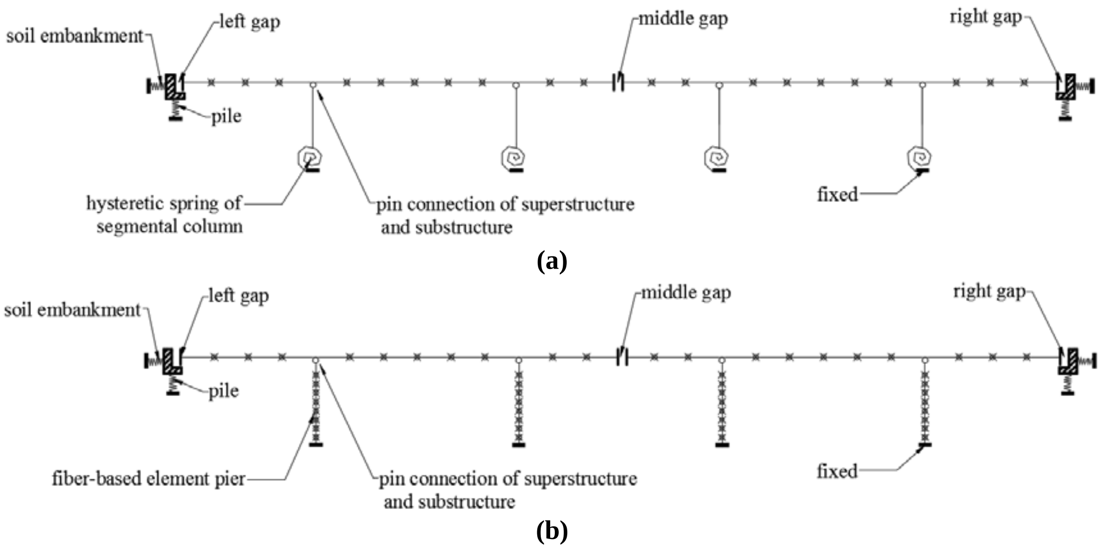


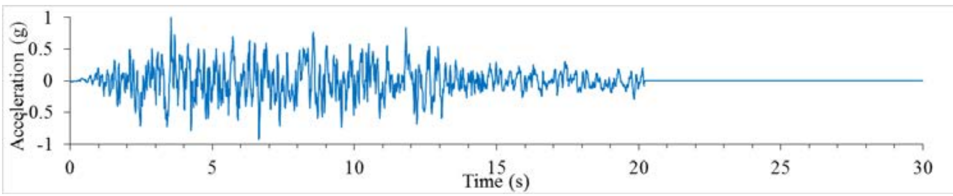
Fig. 3. Finite element models of the bridge structure with (a) segmental columns and (b) monolithic columns

The validated column models are extended to the whole bridge structures to study the seismic responses. Figs. 3(a) and 3(b) show the finite element models of the bridge structure supported by segmental and monolithic columns respectively. The bridge girder is modelled by the 2D elastic beam-column elements, and no plastic deformation is considered. The poundings between the bridge girders (at the middle gap) and between the bridge girder and corresponding abutment (at left gap and right gap) are considered and they are modelled by the Kelvin impact model (Wolf and Skrikerud 1980) in the present study. The behaviours of the abutment piles and soil embankment are also considered in the numerical model and they are modelled by non-linear spring elements respectively. More detailed information regarding the numerical modelling can be found in Bi et al. (2017).

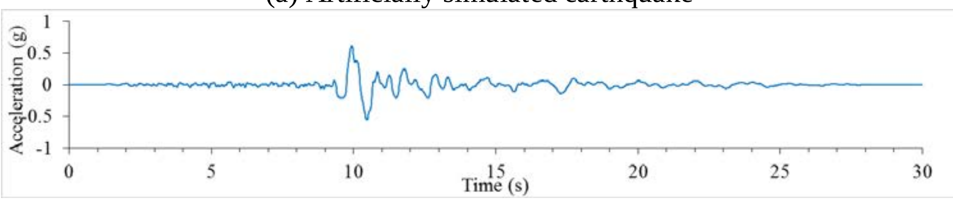
After the numerical models are developed, vibration periods and vibration modes of the bridge system with different columns can be calculated by carrying out an eigenvalue analysis. It is found that the fundamental periods of one bridge frame (either the left or right frame in Fig. 2) are 1.075 s and 1.323 s when the bridge is supported by the monolithic columns and segmental columns respectively. The fundamental vibration modes are the same for the two bridge structures, and it is dominated by the longitudinal movement of the bridge girder. Damping can influence the structural response in the nonlinear time history analysis of structures. Modal damping is applied in the present study and the damping ratio for each vibration mode of two kinds of bridges is assumed as 5%.

4. EARTHQUAKE LOADINGS

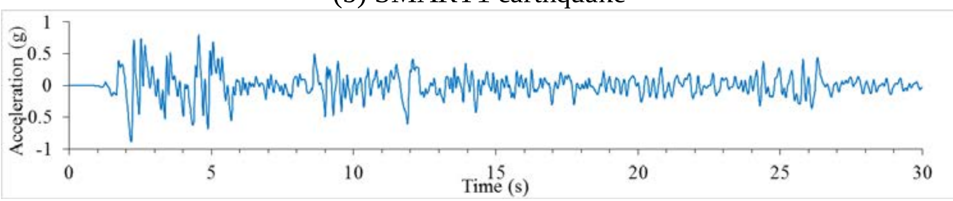
Without loss of generality, three earthquake loadings which have different frequency contents are considered in the present study. Fig. 4(a) shows an artificially simulated earthquake, which is compatible with the design spectra specified in the New Zealand Earthquake Loading Code. The peak ground acceleration (PGA) is assumed as 1.0g. Fig. 4(b) shows an earthquake time history (Event 40, 1986) recorded from the SMART1 array, and it is used to represent a near-fault ground motion. The PGA is scaled to 0.6g in the present study. The last one was recorded from the 1940 El-Centro earthquake (Fig. 4(c)), which represents a far-field earthquake with the PGA scaled to 0.8g.



(a) Artificially simulated earthquake



(b) SMART1 earthquake

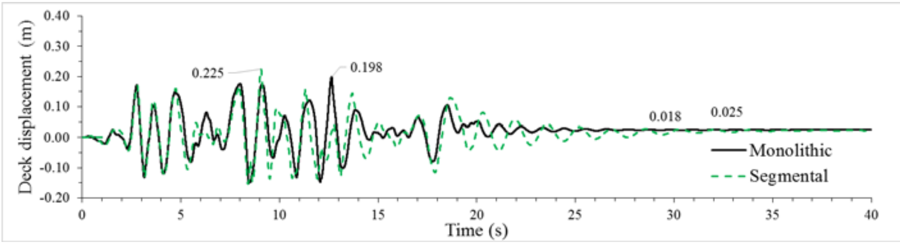


(c) El-Centro earthquake

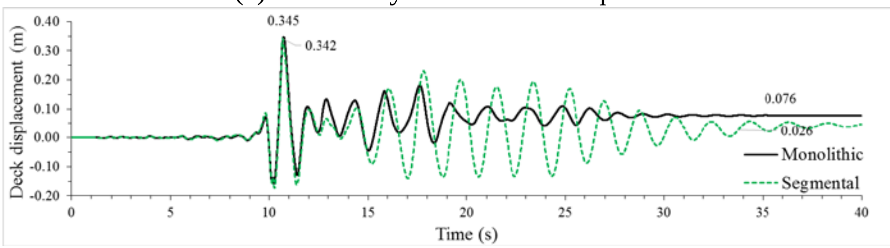
Fig. 4. Different earthquake loadings

5. NUMERICAL RESULTS

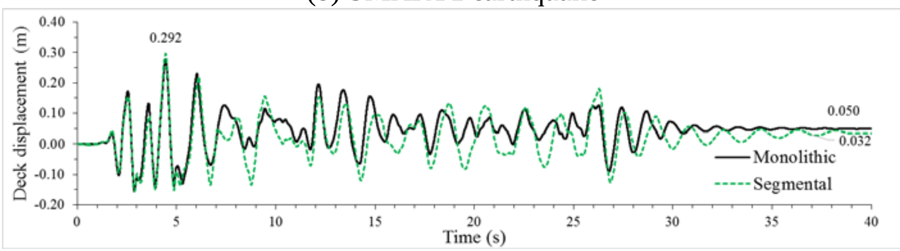
The seismic responses of the bridge structures supported by the segmental columns and monolithic column are investigated in this section. Only the results of the left bridge frame are presented and discussed due to the symmetrical arrangement of the bridge structure. To capture the residual displacement, the simulations are carried out until the bridge structure becomes still. Fig. 5 shows the longitudinal deck displacement of the bridge frame supported by different columns. As shown in Fig. 5, the bridge structure almost stops vibrating when the time reaches 40 sec under these three earthquake loadings.



(a) Artificially simulated earthquake



(b) SMART1 earthquake



(c) El-Centro earthquake

Fig. 5. Deck displacement time histories of the left bridge frame under different earthquake loadings

It can be seen from Fig. 5 that the residual displacements are quite small for the bridges supported by both columns types, and the values from the bridge with segmental columns are slightly smaller than those from the traditional monolithic bridge. The benefit that segmental column can result in smaller residual displacement seems not obvious. However, it should be noted that these results are obtained from the numerical model shown in Fig. 2, in which the size of the separation gaps is 0.1 m each, and the poundings between bridge girders and between the bridge girder and corresponding abutment are considered. In other words, the bridge frames are restrained and cannot vibration freely during the earthquakes. When poundings are not considered in the numerical simulation (as adopted in many previous studies), the residual displacement from the bridge with segmental columns can be much smaller than that from the bridge with monolithic columns. For example, it is reported in Bi et al. (2017) that the residual displacements are 0.03 and 0.156 m respectively for the bridges supported by segmental and monolithic columns respectively under the artificially simulated earthquake loading when the bridge frames shown in Fig. 2 can vibrate freely. This is because when the bridge can vibration freely, the rebars in the monolithic columns may yield during the severe earthquake, while the segmental column can go back to its original position due to the self-centring capability provided by the pre-stressed tendons in the segmental column. These results, on the other hand, also demonstrate the importance of considering pounding in the numerical simulation.

Fig. 5 also shows that the bridge structure with segmental columns experiences more severe super-structural vibrations compared to the one supported by the monolithic columns. This is because less energy is dissipated by the segmental columns compared to the monolithic columns as revealed in many previous studies (e.g. Ou et al. 2007). More energy therefore transfers to the super-structure and results in the more severe vibrations. This may be regarded as a disadvantage of segmental bridge.

Fig. 6 shows the pounding force time histories at the middle expansion joint under different earthquake loadings and Table 2 tabulates the number of poundings. It can be seen from Fig. 6 that there is no obvious trend for the pounding force time histories. The pounding forces in the segmental column case can be larger or smaller than those of the monolithic bridge. For the number of poundings, Table 2 shows that when the bridges are subjected to the artificially simulated earthquake, the bridge with segmental columns results in more number of poundings compared to the bridge supported by the segmental columns. While for the other two earthquake loadings, opposite trend is observed. This is because, as shown in Fig. 7(a), the bridge columns are almost within the elastic range when they are subjected to the simulated earthquake loading. The bridge with monolithic columns are stiffer than that with segmental column as mentioned above, the stiffer structures are usually associated with more number of poundings because they vibrate faster as reported in many previous studies (e.g. Chouw and Hao 2008). Under the SMART1 and El-Centro earthquake loadings, both bridge columns exhibit certain extent of inelastic deformation and less energy is dissipated by the segmental columns as shown in Figs. 7(b) and 7(c), which results in more seismic energy being transferred to the bridge girders and leads to the more violent vibrations of bridge girders as discussed above.

6. CONCLUSIONS

Seismic induced pounding responses of a bridge structure supported by the segmental columns or monolithic columns are numerically investigated in the present study. Numerical results reveal that:

1. The bridge structure supported by the segmental columns can result in more violent superstructure vibrations compared to the one supported by the monolithic columns.
2. When the columns are deform within the elastic range, bridge structure supported by monolithic columns leads to more number of poundings compared to the segmental column case. When inelastic deformations occur, the opposite trend will be observed.

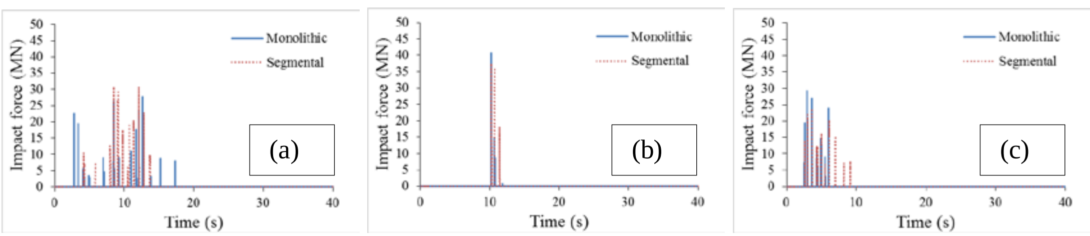


Fig. 6. Pounding force time histories at the middle expansion joint under different earthquake loadings. (a) Artificially simulated earthquake, (b) SMART1 earthquake and (c) El-Centro earthquake

Table 2 Number of poundings at the middle expansion joint when the bridge structures are subjected to the three earthquake loadings

Bridge type	Simulated	SMART1	El-Centro
Monolithic	13	2	7
Segmental	11	3	10

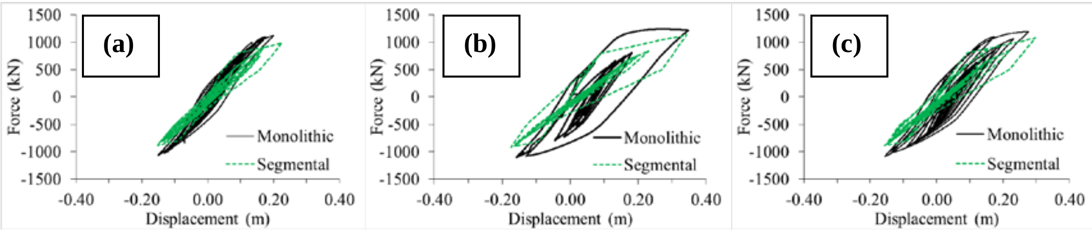


Fig. 7. Force-displacement relationships of different bridge structures. (a) SMART1 earthquake, (b) El-Centro earthquake and (c) artificially simulated earthquake

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