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Double-Curvature Test of Reinforced Concrete Columns Using Shaking Table: A New Test Setup

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Abstract

This paper proposes a new test setup to study the double-curvature behavior of reinforced concrete (RC) columns using shaking table. In this setup, the seismic action is simulated by the horizontal movement of a long-heavy rigid mass sitting on the top of only one test specimen. The double-curvature mechanism of specimen is affected by the movement of the concrete mass on a test rig consisting four steel hollow-section columns fully anchored to the shaking table. Application of axial load on the specimen is made possible through a pre-stressing equipment connecting to its top and bottom bases. The current setup offers two improvements over the previous ones. First, it makes available greater ranges of test data for conducting bigger sizes of the specimens. Second, it allows to directly measure the variation of axial force in the test specimens while the test implementation can be fast and easy with a high safety margin even until the complete collapse of the test units. The current test setup has been successfully applied on two ½ scaled V-shaped columns. It has been shown that the column specimen with a low axial load level of $0.05f'_cA_g$, where f'_c is the concrete strength and Ag is the cross-sectional area of the specimen, can well survive at a ground peak acceleration up to 5.5 (m/s^2) with a drift ratio of approximately 2.91%. Meanwhile, the column subjected to moderate axial load level of $0.15f'_cA_g$ can survive at a higher ground peak acceleration of 8.0 (m/s^2) with a drift ratio of 3.75%. Furthermore, it is experimentally evidenced that the V-shaped cross-section does not deform in-plane under seismic action. The angle between two planes corresponding to the column web and flange are up to 0.03 (rad). This finding is significant since it contradicts the plane strain assumption available in the current design practice.

Keywords: Double-Curvature Test Setup; Seismic Simulation; Shaking Table; Reinforced Concrete V-Shaped Columns.

1. Introduction

It has long been recognized that columns in low-rise RC buildings, especially those at the first floor, deform and finally fail in double-curvature shape under seismic events [1-3]. The double-curvature failure of columns often causes the whole building structures unstable or even collapse, resulting in damage and loss of lives. Therefore, it is not surprising that this mechanism of RC columns has been extensively tested in both quasi-static and dynamic loading conditions. The quasi-static tests have been successfully applied on both small scale and large scale column specimens with a variety of cross-sectional shapes [4, 5].

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The magnitude of axial force applied on the test column that represents gravity loads during a seismic event can be set as high as $0.05f'_cA_g$, where f'_c is the concrete strength, and A_g is the cross-sectional area of the specimen. Also, this magnitude can be variable depending on the test objectives. Meanwhile, most of previous dynamic tests, so-called shaking table tests, often conducted on small scale specimens due to the limited sizes and load capacities of the shaking table. To form the double curvature mechanism, it is required at least two specimens that are fully fixed to the table at their bottom bases and to a rigid mass on their top bases to be tested at a time [1-7]. Axial forces applied on the specimens are at a relatively low level, say $0.1f'_cA_g$, and are not changeable during the test run since these forces are supplied by the weight of the rigid mass. It is worth noting that high but varying levels of axial forces are an important feature of RC columns integrated in the dual (frame/wall) structural system that is widely used in the modern buildings.

In the past, the seismic behavior of RC building structures has been extensively studied all over the world with a great range of approaches and objectives. In terms of experimental investigation, the test specimens were extracted from either one typical story [1-7] or from multi-storey [8-10]. The test specimens could be either an isolated RC wall with various types of cross-sectional shapes such as I-, C- and U- shaped [11-14] or RC frames with various seismic detailing [15, 16]. Effects of various types of ground motions and bearings on the overall seismic performance of building structures have also attracted great attention from the seismic research community [17-20]. In terms of methodology, besides the experimental methods that often employ a shaking table, the theoretical approaches such as numerical simulation [21-23] and probabilistic seismic assessment [24] play an important role in the research area.

This research proposes a new experimental approach to study the double-curvature behavior of RC columns using shaking table. In each test, only one specimen, instead of two or three units, is tested to maximize its size, allowing a better understanding of the test performance. The sliding mechanism is enabled by a test rig consisting of four steel hollow section columns fully fixed on the shaking table and a rigid mass sitting on the top of the specimen. A pre-stressing system externally setup in line with the specimen to supply the expected levels of axial force. To demonstrate the application of this setup, two tests on ½ scaled V-shaped columns with two levels of axial load, $0.05f'_cA_g$, and $0.15f'_cA_g$, are presented. The potentiality and limitations of the new experimental approach together with several main aspects of the seismic performance of V-shaped columns such as crack patterns and effects of axial force on the final failure modes are also discussed.

2. Proposed Experimental Setup For Double-Curvature Test of RC Columns

Figure 1(a) shows a schematic view of the double curvature test for one column specimen using shaking table. The test specimen is fully fixed to the shaking table and pre-stressed by a compression force P. A rigid mass (weight W) is attached to the top of the specimen and to two rollers A and B. When the rigid mass moves horizontally to the left side, it tends to rotate counter clock wise but restrained from doing so by roller A, creating reaction force R. This reaction force will result in bending moment M and axial force N in the test specimen by two following equations:

$$M = RL \tag{1}$$

$$N = P + W - R \tag{2}$$

The diagrams M and N are shown in Figure 1(b) and (c).

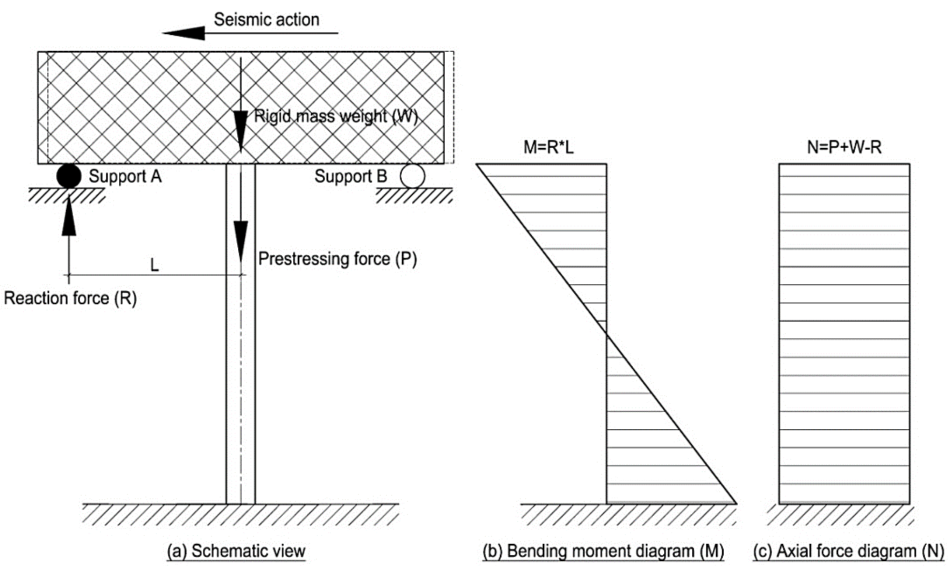
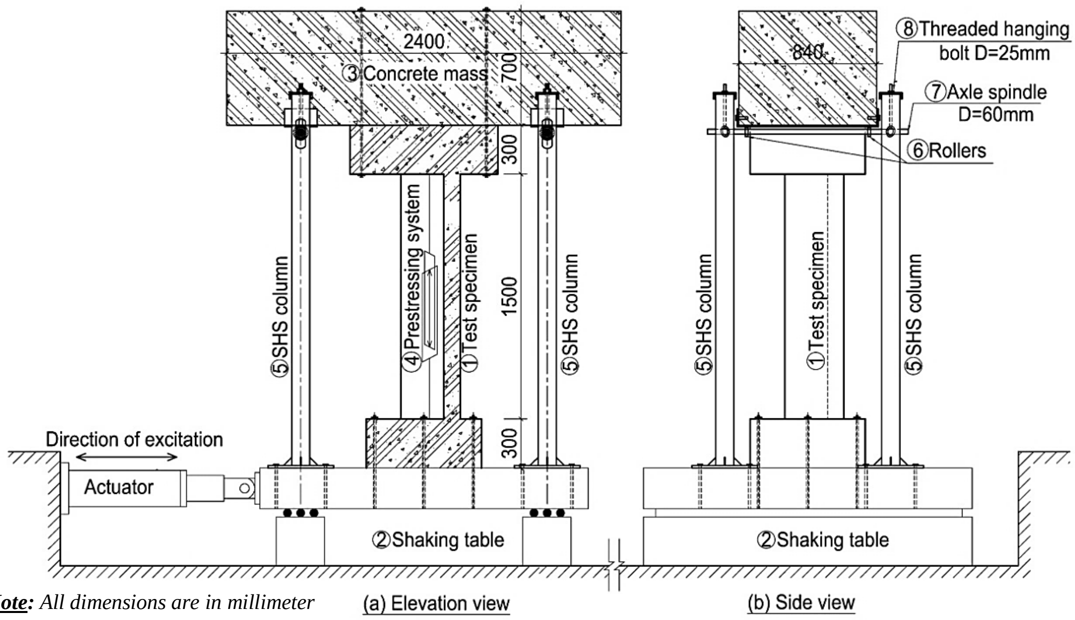


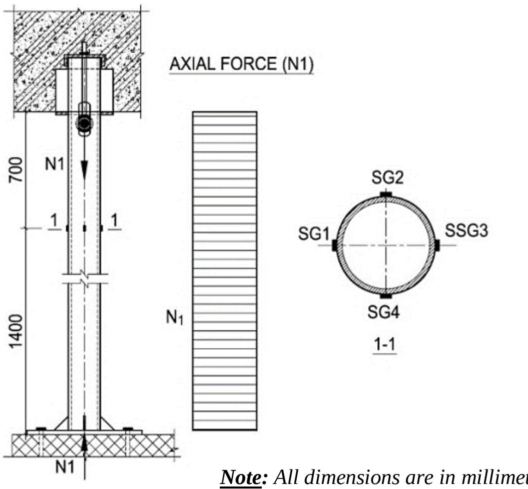
Figure 1. A schematic view of sliding mechanism



Note: *All dimensions are in millimeter*

Figure 2. Detail of the test setup

Figure 2 provides the design of the test-setup for a ½ scaled V-shaped cross-sectional column specimen comprising 8 elements denoted from 1 to 8. The test specimen (denoted as 1) is fully fixed to the shaking table (2) through its bottom concrete base, and to concrete mass (3) through its top base by sets of high-strength bolts. The total height of the specimen is 2100 (mm), and dimensions of the concrete mass are 2400 mm × 840 mm × 700 mm with a weight of 35 kN. A pre-stressing force P in the test specimen is provided by a pre-stressing system (4), while vertical supports A and B are formed by four Steel Hollow Section (SHS) columns (5) that are fixed on the shaking table at both sides of the specimen along the direction of excitation, together with the rolling systems (denoted as 6, 7, and 8).



Note: *All dimensions are in millimeter*

Figure 3. A measuring method for axial force in SHS columns

In order to measure the reaction force R and so the axial force in the test specimen N (as illustrated in Figure 1), each SHS column (D114 mm × 3.5 mm) is mounted with four strain gauges at its cross-section 700 mm away from the top end, as shown in Figure 3. The axial force in the column can be calculated using the recorded steel strains and the cross-sectional properties as follows:

$$N_1 = E_s A_s (\varepsilon_1 + \varepsilon_2 + \varepsilon_3 + \varepsilon_4)/4 \tag{3}$$

Where: A_s is the cross-sectional area of the hollow steel column; E_s is Young's modulus of steel; ε_1, ε_2, ε_3, and ε_4 are the recorded steel strains at Section 1-1.

It is worth noting that this measuring method has been successfully applied for both quasi-static and dynamic tests [25-27].

Figure 4(a) and (b) show the schematic and actual views of the pre-stressing system (4) which is connected in-line with the test specimen. The system consists of two steel frames, one is attached to the top base and another is attached to the bottom base, and a hydraulic jack (together with a load-cell) placed in the between. During the test run, the pre-stressing force P can be kept constant at desired levels by a manually controlled oil pressure station (Figure 4(c)).

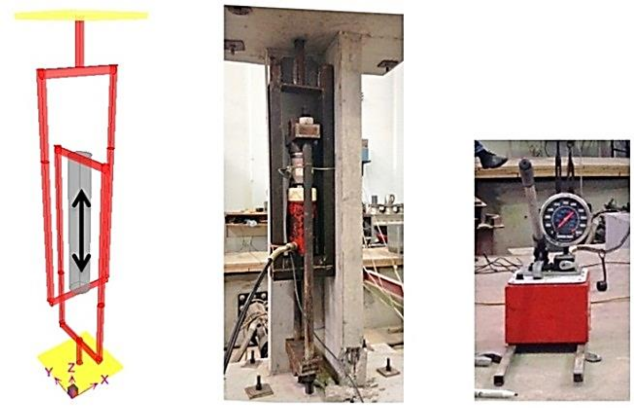


Figure 4. Detail of pre-stressing system (4)

Figure 5 provides a photo of the test setup during the preparation process. In this experimental program, it is proposed that the column specimens are tested by the same simulated ground acceleration record with increasing levels of the peak values. In each test corresponding to each value of the peak ground acceleration, the parameters of interest that are recorded (other than the pre-stressing force P and axial force N1 in SHS columns mentioned above), including: (*i*) acceleration of the rigid mass by accelerator ACC1 that will be used to calculate the seismic force, (ii) horizontal displacement of the mass along the direction of excitation by two linear variable differential transformers (LVDT) placed at both ends of the mass, and (iii) concrete and reinforcement strains by strain gauges mounted on the specimen. After each test, any arising gap between the rigid mass and the rolling system is cleared by tightening the hanging threaded bolts (8). The specific locations of strain gauges mounted on the specimen will be shown in the next section.

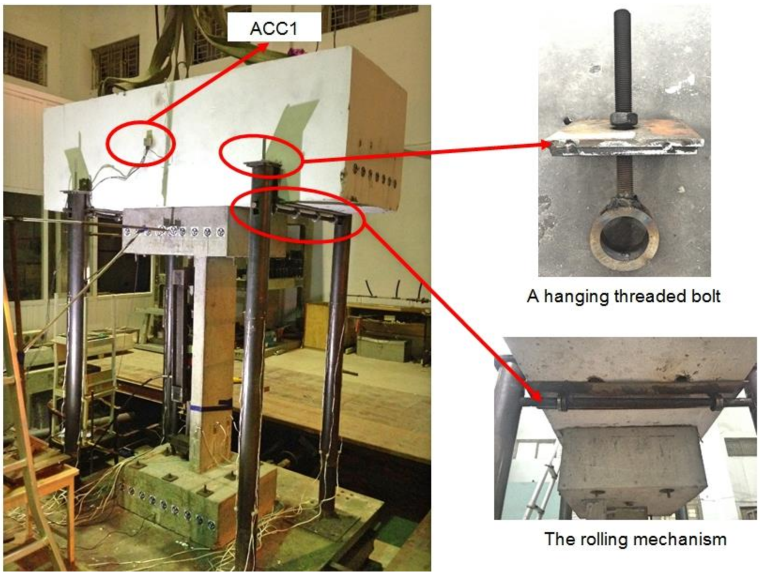


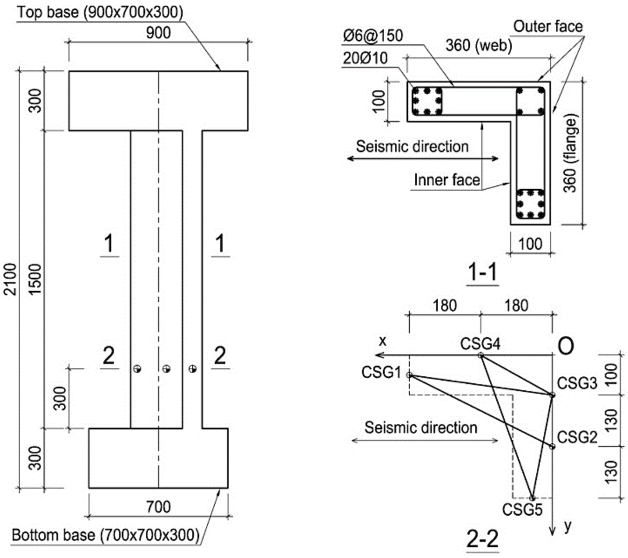
Figure 5. A photo of the test setup

3. The Double-curvature Behavior of V-shaped Columns

3.1. Input Data

The proposed experimental approach is applied to two double-curvature V-shaped column specimens with two specific objectives as follows. The first is to examine the seismic behavior of such columns subjected to two levels of gravity axial loads: a low level of 5% f'_cA_g (50 kN) when the column is located at the corner of a prototype low-rise building (Specimen S1), and a medium-low level of 15% f'_cA_g (150 kN) when the column is located at the perimeter edge of the building (Specimen S2). The second objective is to examine the extent of application of plane strain assumption, which is currently adopted for reinforced concrete sectional design of this type of structural elements in most codes of practice [28, 29].

Figure 6 provides the reinforcement detail of two specimens, whose dimensions are scaled by a factor of ½ from the prototype ones while reinforcement ratios are kept the same. Both column web (along the excitation direction) and column flange (perpendicular to the excitation direction) has a length of 360 (mm) and a thickness of 100 (mm). The clear height of the columns is 1500 (mm). The specimen cross-section is reinforced with 20 longitudinal bars, each with a diameter of 10 (mm) (longitudinal reinforcement ratio of 2.5%), and 6mm-diameter stirrups with a spacing of 150 (mm). To fulfill the second research objectives, each specimen is mounted with five concrete strain gauges at the column section 300 (mm) away from the bottom base. The strain gauges are denoted as CSG1, 2, 3, 4, and 5 as shown in Figure 6.



***Note**: All dimensions are in millimeter*

Figure 6. Details of specimens

Concrete used in the test was a small-aggregate mix with the actual strength of 20.0 MPa and the secant elastic modulus of concrete was 27000 MPa. The average yield and ultimate strengths of both the longitudinal and the transverse reinforcing bars were 365 MPa and 390 MPa, respectively.

Two specimens were tested to complete failure using the acceleration time history of Tolmezzo earthquake, as shown in Figure 7, with a time duration of 15 seconds and a damping ratio of 5%. The record is applied with seven levels of the peak ground acceleration, namely EQ1 to EQ7, whose actual values recorded during the test run are presented in Table 1.

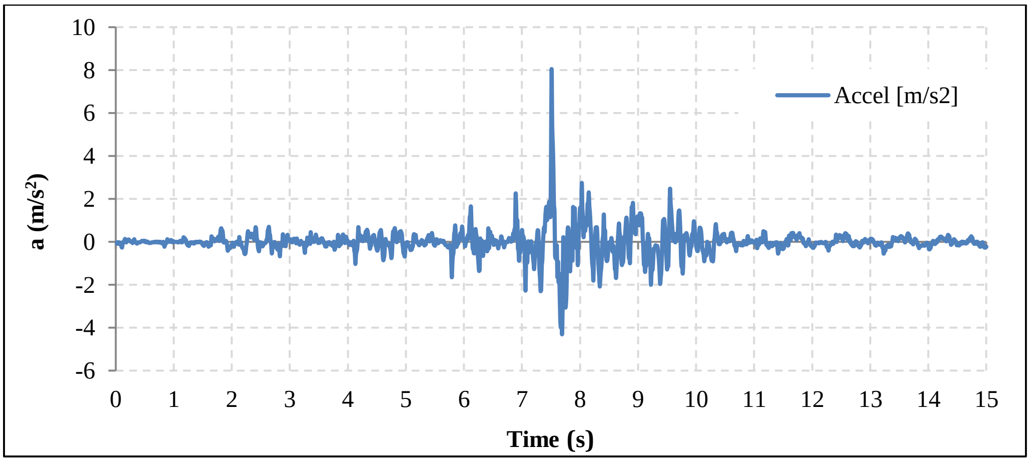


Figure 7. The simulated acceleration time history

Table 1. The actual values of the peak ground acceleration according to the seven tests

Test	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Peak ground acceleration (m/s^2)	1.3	1.9	3.4	4.1	5.5	6.2	8.0

3.2. Crack Development and Failure Modes

No cracks appeared on both specimens during Test EQ1, but as illustrated in the following section, these specimens damaged to some extent or even collapsed at certain values of ground peak acceleration. Figures 8 (a), (b), (c), and (d) respectively show the crack patterns of Specimen S1 at Tests EQ2, 3, 4, and 5, and Figures 9 (a), (b), (c), (d), and (e) respectively show the crack patterns of Specimen S2 at Tests EQ3, 4, 5, 6 and 7. A clear tendency in the formation and development of cracks can be observed is that the initial cracks only appeared on the outer face of the column. With increasing ground peak acceleration, the cracks increased significantly in quantity and spread to the inner face as well as from the corner to the outside of the column. There have been some significant differences between the crack patterns of Specimens S1 and S2 as well as their failure modes. The observations of the structural response of Specimen S1 when being subjected to the combination of seismic load and low axial load indicated that the column collapse was mainly due to high shear stresses and the increasing of the seismic load, accompanied by a clear change from the flexural to the shear performance of the column. Firstly, when the input acceleration was low (EQ2), only horizontal cracks were observed on the outer face of the flange. Meanwhile, there was no observed concrete cracking on the column web (refer to Figure 8(a)). At higher values of the ground peak acceleration, multi diagonal cracks appeared on the strong flange that progressively developed into the inner face of the column (as shown in Figure 8(b), (c), (d)), and eventually caused the column’s collapse. In contrast, for Specimen S2, with a moderate axial load level, the horizontal cracks due to the flexural action accounted for the majority of observed cracks during the test; and there were only two hairline diagonal cracks observed at the failure state (as shown in Figure 9(e)). That lead to a strong belief that there was no change in the working state for Specimen S2 and its collapse was mainly due to the combined axial force and bending moment resulted from seismic action.

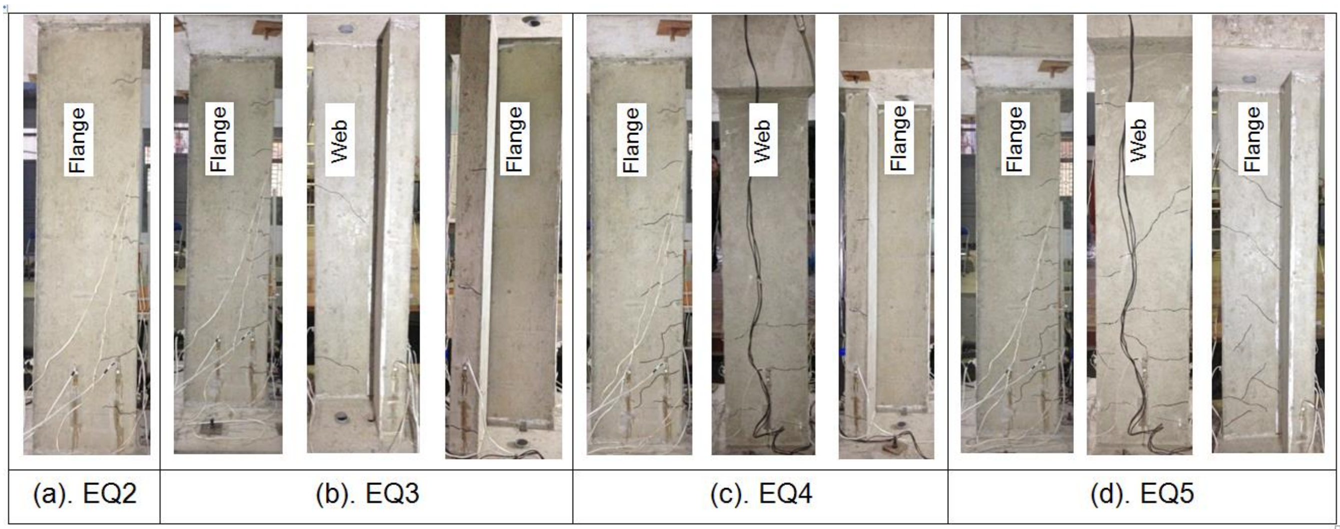


Figure 8. Crack patterns of Specimen S1 at Test EQ2, 3, 4, and 5

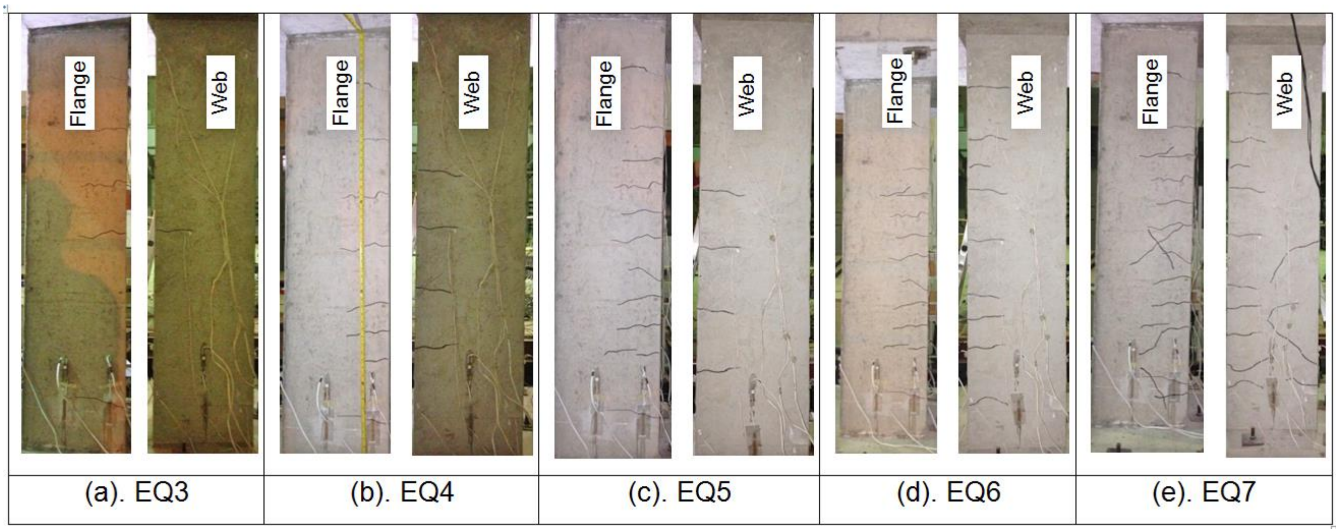


Figure 9. Crack pattern of Specimen S2 at Test EQ3, 4, 5, 6 and 7

The speed of the crack development leading to the failure of the Specimen S1 was also considered significantly faster than specimen S2. Specimen S1 collapsed early at EQ5 when almost of the cracks developed into the inner face of the column and the story-drift only reached 2.91%. Meanwhile, for the Specimen S2 the first cracks were only observed after the EQ3 and almost were not spread to the inner face of the column. The specimen was only collapsed at EQ7 with a story-drift of 3.75%, and its residual strength and stiffness was considered to be still significant to resist the seismic action. As shown in Figure 10, it can be seen that the story-drift of the specimen S2 was always a little smaller than the Specimen S1's one throughout the tests.

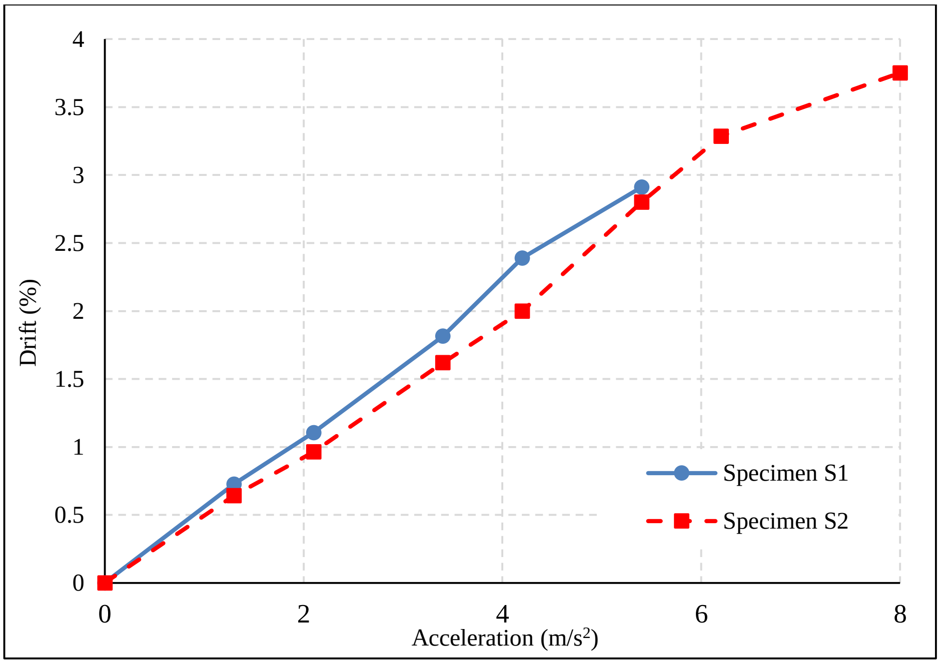


Figure 10. The relationship between drift ratio and ground peak acceleration of Specimen S1 and S2

Figure 11 presents the final failure state of the two specimens.

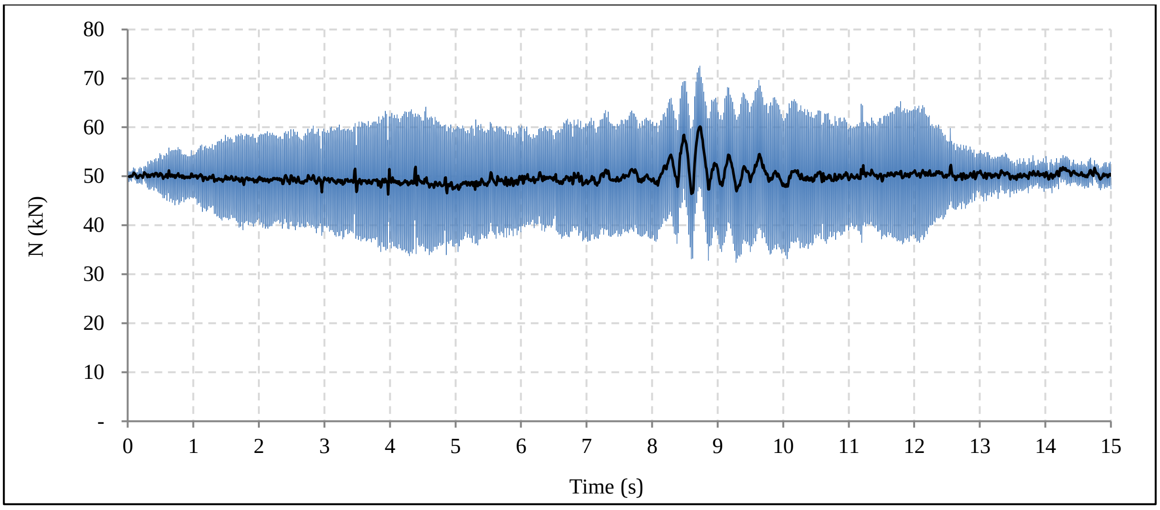
Figure 11. The collapse states of two specimens

3.3. Variation of Axial Forces in the Test Columns

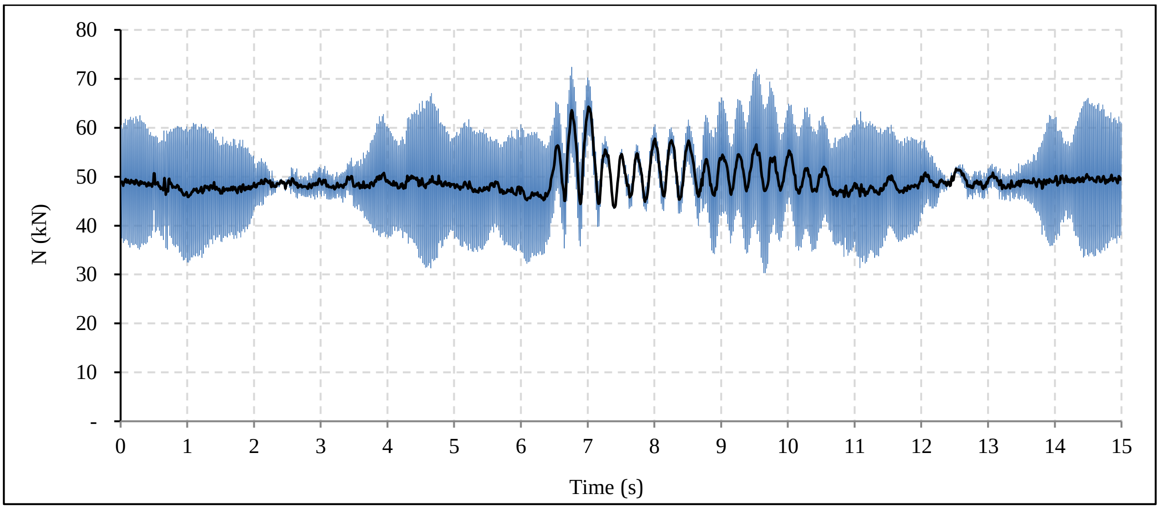
The variation of axial force N in a double-curvature column subjected to seismic excitation is an important factor to investigate its seismic performance. The axial force N is calculated by Equation 2 and shown in Figure 1. In the proposed test setup, the reaction for R can be determined by the following equation:

$$R = N_1 + N_2 + N_3 + N_4 \quad (4)$$

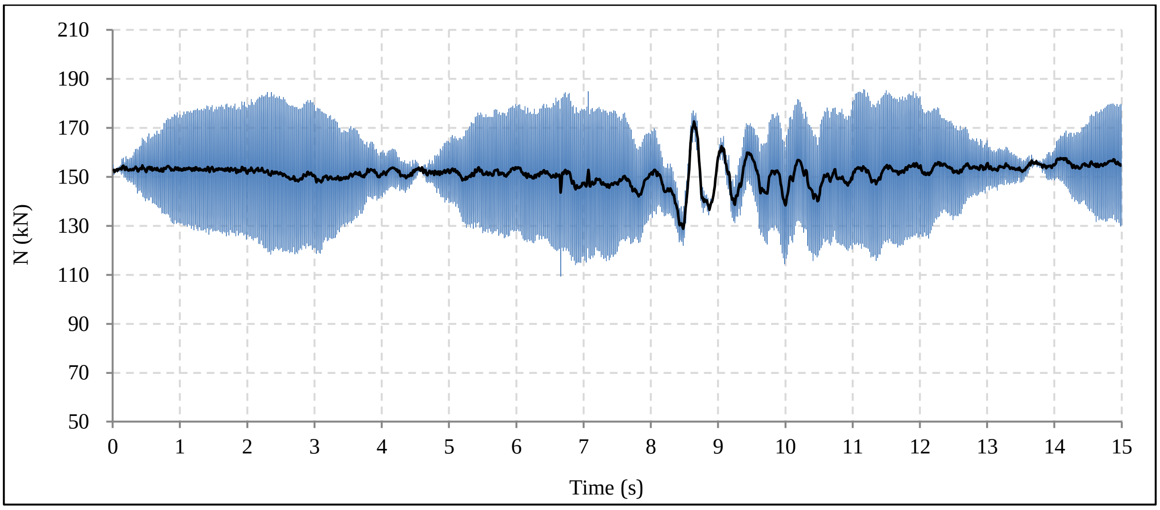
Where N_1, N_2, N_3, and N_4 are the axial forces in four surrounding SHS columns that are calculated based on the corresponding strain gauge readings through Equation 3.



(a) Specimen S1 at EQ4



(b) Specimen S1 at the last test EQ5



(c) Specimen S2 at the last test EQ7

Figure 12. The variation of axial force in Specimens S1 and S2

Figure 12 shows three selected time history profiles of axial force N in two Specimens, including Specimen S1 at EQ4 and at the last test EQ5 (Figure 12 (a) and 12 (b), respectively), and Specimen S2 at the last test EQ7 (Figure 12c). As can be seen in Figure 12 (a), the variation of axial force N in the working state can be as high as +/- 25% of the mean (static) value. Meanwhile, at the collapsed state, the variation tends to increase up to 50%, as can be shown in Figure 12 (b) and (c). It is worth noting that the positive variation above the mean (static) value of axial load is resulted from the downward vertical acceleration arising during the horizontal excitation by the shaking table. A sudden decrease in the mean value of axial force N in Specimen S2 at the last test EQ7 indicates that the specimen partially losses its axial strength and is supported by the SHS columns.

Examination of plane strain assumption for V-shaped cross-sectional columns

In the current versions of two well-known codes of practice [28, 29], plane strain assumption is adopted for the cross-sectional design of vertical structural members subjected to combined axial force and bending moments resulted from seismic action. This assumption is certainly best suited for columns with traditional cross-sectional shapes (e.g. square, circle and rectangular), and is further extended for walls and irregularly shaped columns such as I-, V- and U-shaped. However, in the recent tests on V-shaped columns under seismic action [3, 4] it has been observed a longitudinal crack along the column height separated their two flanges, which in some cases developed to the final splitting failure mode. It is expected such type of crack may affect the plane strain distribution of the column cross-sections.

To examine the validity of this design assumption, each of two test specimens in this investigation is mounted with five concrete strain gauges located at the same column section away 300 (mm) from the bottom base (Figure 5). The concrete data together with coordinates of the strain gauge locations are used to create two planes, Plane P with three reference points CSG1, 2, 3 and Plane Q with CSG3, 4, 5, as summarized in Table 2. Planes P and Q are defined via points having coordinate shown in Table 2.

Table 2. The coordinate of points on planes P and Q

Plane		**P**	**Q**	**P/Q**	**Q**	**Q**
Point		CSG1	CSG2	CSG3	CSG4	CSG5
Coordinate	x_i	360	0	0	180	50
	y_i	50	230	100	0	360
	z_i	ε_1^*	ε_2^*	ε_3^*	ε_4^*	ε_5^*

The cosine of the angle between plane P and Q is determined through the direction vector of these planes as the following equation:

$$cos\varphi\left|\cos(\overrightarrow{n_P},\overrightarrow{n_Q})\right| = \frac{\left|\overrightarrow{n_P}.\overrightarrow{n_Q}\right|}{\left|\overrightarrow{n_P}\right|.\left|\overrightarrow{n_Q}\right|} \tag{5}$$

Where: $\overrightarrow{n_P}$ and $\overrightarrow{n_Q}$ are the direction vector of plane P and Q and having the coordinate as follows:

$$\overrightarrow{n_P} = (n_{P1}, n_{P2}, n_{P3}) \tag{6}$$

$$n_{P1} = (y_2 - y_1)(\varepsilon_3^* - \varepsilon_1^*) - (y_3 - y_1)(\varepsilon_2^* - \varepsilon_1^*) \tag{7}$$

$$n_{P2} = (x_3 - x_1)(\varepsilon_2^* - \varepsilon_1^*) - (x_2 - x_1)(\varepsilon_3^* - \varepsilon_1^*) \tag{8}$$

$$n_{P3} = (x_2 - x_1)(y_3 - y_1) - (y_2 - y_1)(x_3 - x_1) \tag{9}$$

$$\overrightarrow{n_Q} = (n_{Q1}, n_{Q2}, n_{Q3}) \tag{10}$$

$$n_{Q1} = (y_4 - y_3)(\varepsilon_5^* - \varepsilon_3^*) - (y_5 - y_3)(\varepsilon_4^* - \varepsilon_3^*) \tag{11}$$

$$n_{Q2} = (x_5 - x_3)(\varepsilon_4^* - \varepsilon_3^*) - (x_4 - x_3)(\varepsilon_5^* - \varepsilon_3^*) \tag{12}$$

$$n_{Q3} = (x_4 - x_3)(y_5 - y_3) - (y_4 - y_3)(x_5 - x_3) \tag{13}$$

Figure 13 and Figure 14 show the time history profiles of the angle between two Planes, P and Q, of the two tested specimens. As can be seen in Figure 13 and 14, both profiles are kept at very low level first, and reached the peak value right after the peak ground acceleration, and then quickly decreased to residual values. Also, with increasing of the peak ground acceleration, the angles generally increase for both specimens.

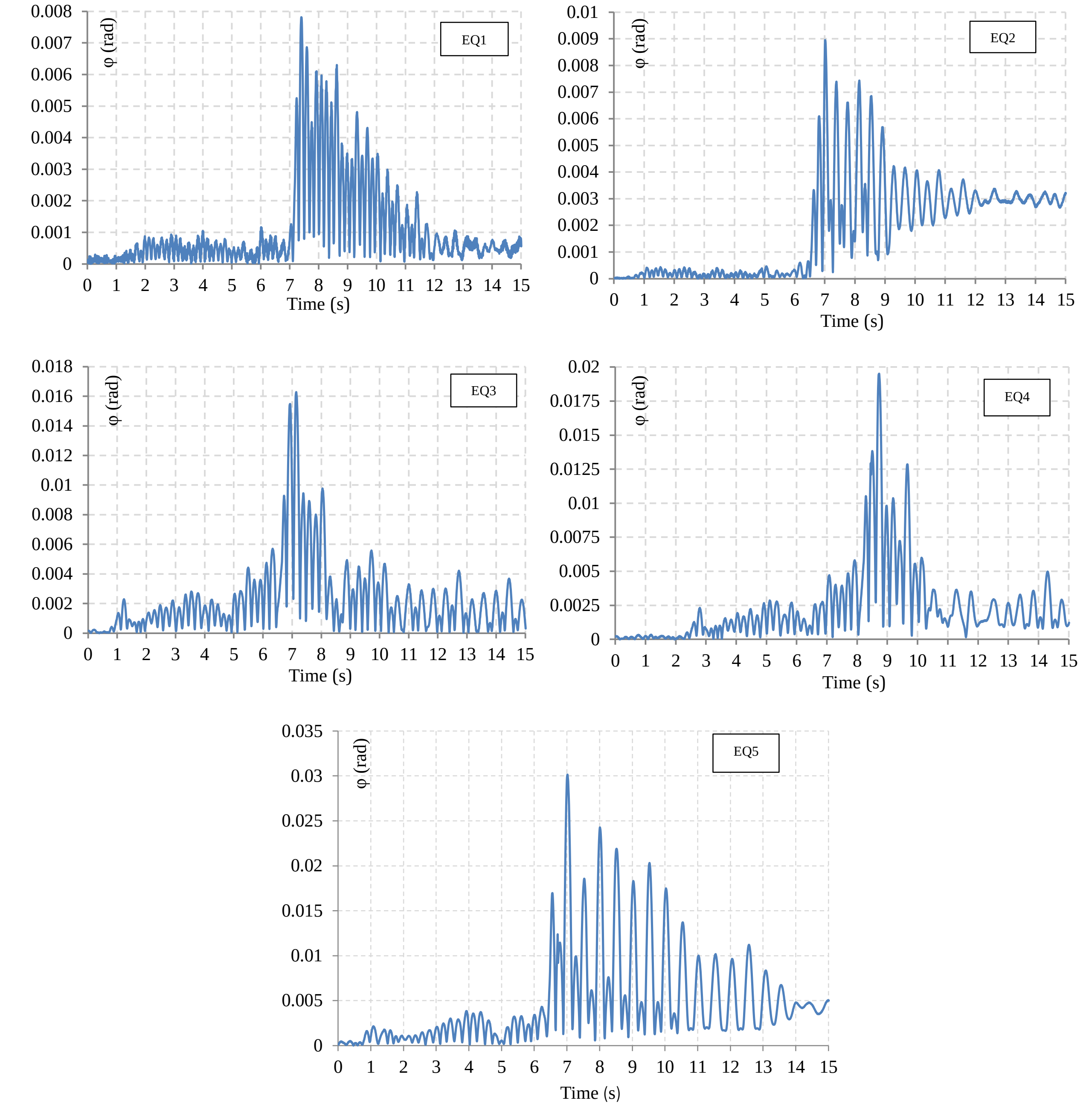
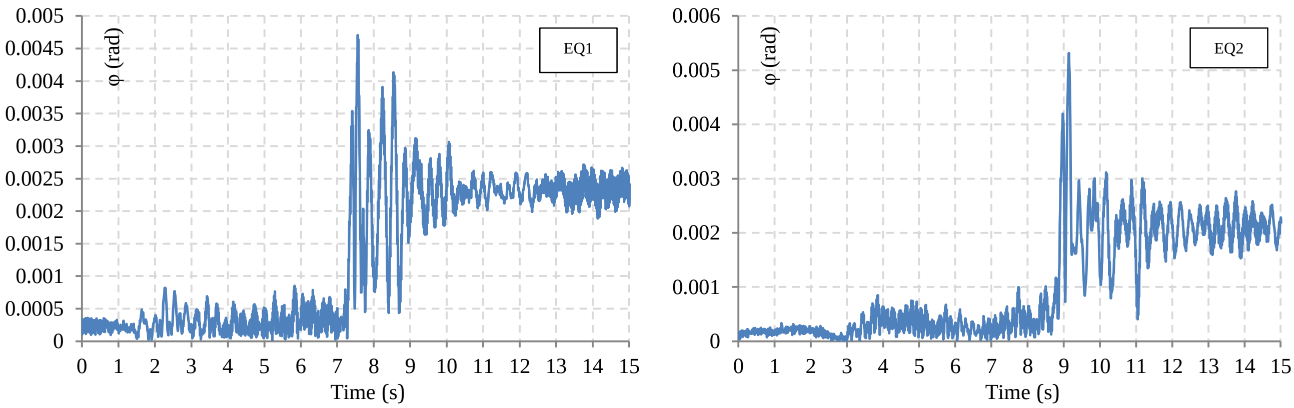


Figure 13. Time history of the angle between Planes P and Q of Specimen S1



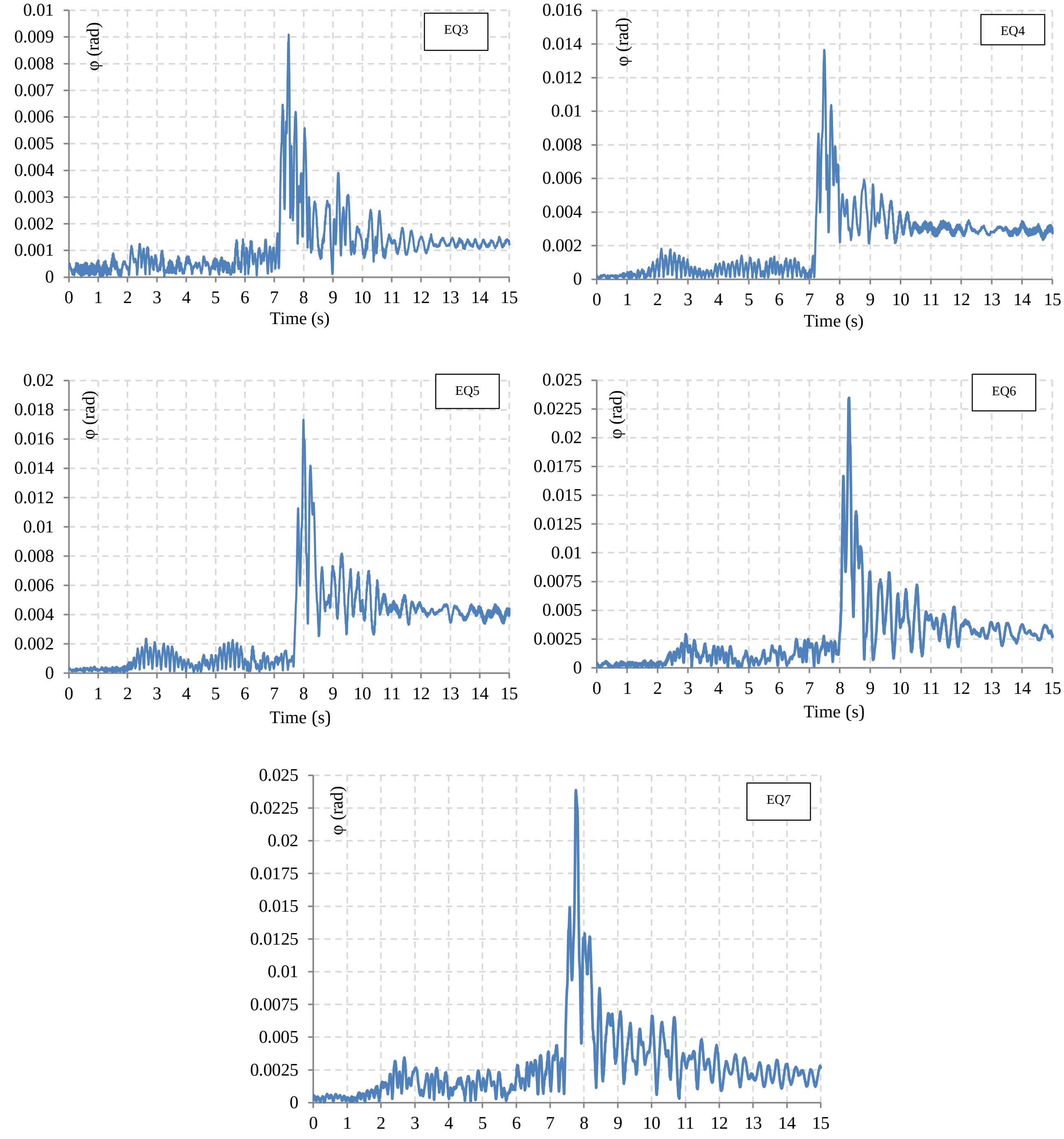


Figure 14. Time history of the angle between Planes P and Q of Specimen S2

Figure 15 compares the profiles of the peak angle between two Planes (P, Q) of Specimen S1 and Specimen S2 according to increasing levels of excitation, from EQ1 to EQ8. Apart from Test EQ1, in which there is no visible difference between the two curves, the peak angle of Specimen S1 is continuously increasing with increasing ground peak acceleration towards the specimen's failure at Test EQ5 where it reaches a value of 0.03 rad. Meanwhile, the peak angle of Specimen S2 increases at a lower gradient until EQ5, then it remains constant at 0.024 towards the last test EQ7. The differences between the two curves are significant, varying from 0.004 rad at EQ2 (equal to 44% that of Specimen S1) to 0.013 rad (43% that of Specimen S1) at EQ5.

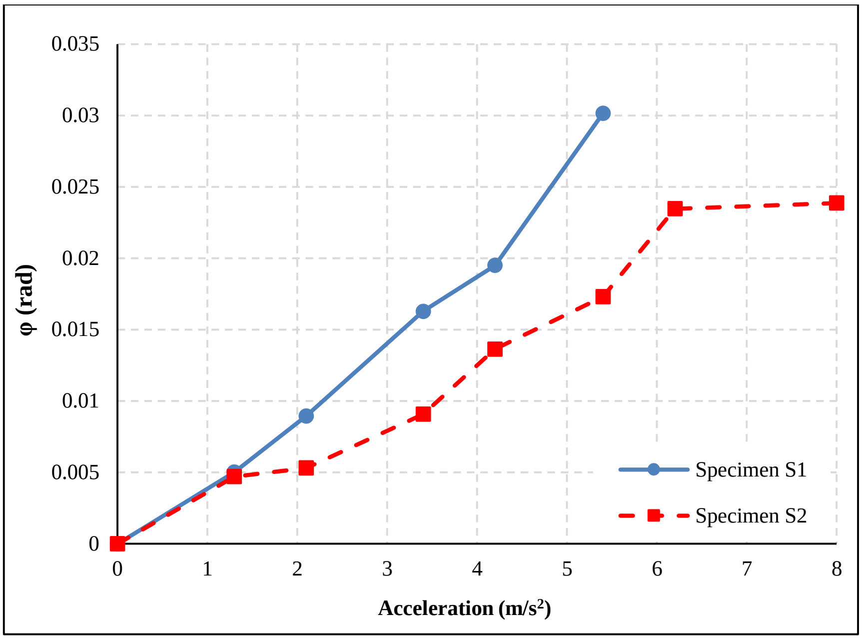


Figure 15. Comparison of the peak angle between two planes (P, Q) for two specimens

It has been long recognized that shear and warping deformations are two sources that possibly affect the plane strain assumption of cross-sectional irregularly shaped column. However, so far there have been limited experimental studies quantifying these two factors. In this investigation, the greater values of the peak angle between plane P and Q of Specimen S1 in comparison to those of Specimen S2, as mentioned above, could be contributed from high shear deformation in tests on Specimen S1, which finally became multi diagonal cracks on its web as shown in Figure 8. The current investigation also shows that greater axial force in V-shaped columns can limit the shear deformation and cracks as well as enhance the validity of plane strain assumption for cross-sectional design.

4. Conclusion

This paper proposes an experimental approach to investigate the seismic behavior of double-curvature RC columns using shaking table. The successful application on two V-shaped cross-sectional specimens has shown that the proposed setup can fairly simulate the desired double-curvature mechanism with various levels of applied axial loads. Besides, for the test setup conducts on only one specimen at a time (instead of two in the previous tests), the size of test specimens can be greater, allowing greater ranges of test data available. Last but not least, the test preparation and implementation can be fast and easy with a high safety margin, even when the specimen completely collapses since four surrounding SHS columns can fully support the collapsed specimen.

The seismic behavior of V-shaped columns has also been presented herein, including the crack pattern, the variations of axial forces and the final failure modes. It has been shown that the V-shaped column with a low axial load level of $0.05f'_cA_g$ can well survive at a ground peak acceleration up to 5.5 (m/s^2) with a drift ratio of approximately 2.9%. The column subjected to moderate axial load level of $0.15f'_cA_g$ has a much better seismic performance, that can survive a higher ground peak acceleration of 8.0 (m/s^2) at which the drift ratio attained a value of 3.7%.

It is experimentally evidenced in this study that the V-shaped cross-section does not deform in-plane when subjected to seismic action as traditionally assumed in design practice. As observed in the current investigation, the values of the angle between two planes corresponding to the column web and flange are as high as 0.03 (rad) and 0.024 (rad) for two respective Specimens S1 and S2. Therefore, further studies are needed to clear the difference between the traditional plane strain assumption and the actual deformation state of such irregularly shaped columns.

5. Funding and Acknowledgements

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6. Conflicts of Interest

The authors declare no conflict of interest.

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