SEISMIC SHEAR STRENGTHENING OF RC COLUMNS BY STEEL ENCASEMENT WITH DIRECT FASTENING CONNECTIONS

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Abstract

Reinforced concrete (RC) columns are crucial structural members that resist both gravity and seismic loads. The seismic resistance of existing RC columns designed based on old concrete design codes may not meet the latest design standard. Hence a new strengthening method that uses a steel encasement to increase the axial and seismic shear capacity and deformability of RC columns with a square section is proposed. This method is quick and convenient because direct fastening is used to assemble the system. In this method, unique hardened fasteners are driven into steel material by a powder-actuated gun or battery-actuated gun. Four columns including two control columns and two strengthened columns have been tested under reversed cyclic loads to examine the reliability and effectiveness of the proposed method. Special attention has been paid to the robustness of the connections joined by direct fastening. Although direct fastening plays a crucial role in the structural performance of the proposed method, current design guidelines have not incorporated this new connection technique. Hence, extensive connection tests have been conducted to evaluate the key factors that affect the bearing strength of the connections. New design equations for the connections that use direct fastening are presented.

Keywords: Seismic shear strengthening; rectangular column; reinforced concrete; steel encasement; direct fastening

1. Introduction

Recent post-earthquake investigation revealed that most reinforced concrete (RC) columns that were designed and built according to the old specifications in which seismic design failed to resist the earthquake force and displacement demands and resulted in collapse [1-4]. This has induced great loss in infrastructures, properties or even lives. There is an urgent need to develop quick, convenient and effective strengthening methods to strengthen the columns of building stocks prior to strong earthquake attack, particularly for low-to-moderate seismicity regions (e.g. Hong Kong and Malaysia) where new earthquake design considerations are to be implemented in practice.

One of the prevalent strengthening strategies for RC columns is RC jacketing [5-7]. Thermou et al. [7] conducted laboratory test on RC jacketed columns. They found that the ductility and strength of the strengthened columns were noticeably enhanced, which is further observed in other studies [5, 6]. The enhancement is attributed to the enlargement of the column section and the newly erected RC jacketing can share partial external load and provide confinement to inner concrete core. Nevertheless, the construction of RC jacketing involving wet-trade is time consuming and labor intensive. Moreover, the indoor available space is reduced, which is not a wise choice for metropolis, like Hong Kong. Furthermore, the interface slip between the old and new concrete impairs the composite action of the strengthened columns [8]. Various construction treatments [9, 10] have been trialed to circumvent this detrimental behavior.

Apart from RC jacketing, high performance fiber reinforced cementitious composite (HPFRCC) was proposed to strengthen RC columns. Li et al. [11] conducted experiments on four columns to examine the effectiveness of HPFRCC. Their results demonstrated that the shear capacity and ductility was increased marginally when compared to the unrepaired columns. Cho et al. [12] improved this method by combining additional steel bars. Although effectiveness is improved, the section is enlarged and extensive work is required to roughen the concrete surface.

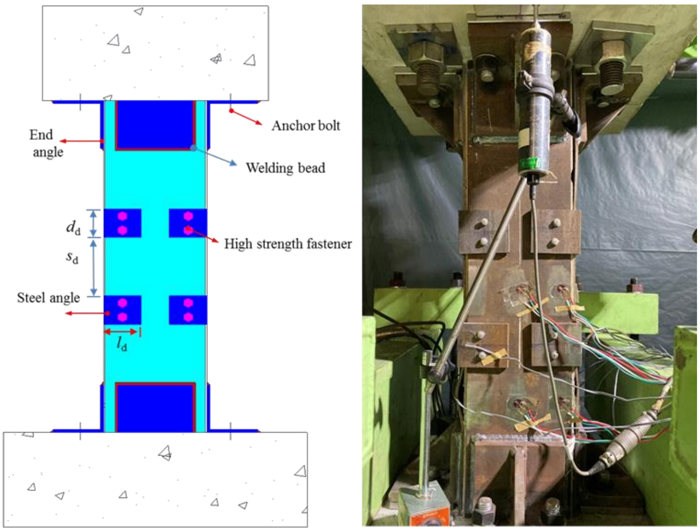


Fig. 1 – Proposed shear strengthening method

To address the aforementioned issues, a nonconventional encasement that comprises four steel plates fixed with steel angles by using the direct fastening method is proposed in this study; see Fig.1. Direct fastening means that unique hardened fasteners are driven into steel material by a powder-actuated gun or battery-actuated gun [13]. In this approach, the proposed steel encasement can directly share the axial load and increase the flexural strength. Also, the direct-fastened connections of the steel encasement behave like transverse reinforcement, which could enhance the shear capacity. When compared to other strengthening methods, the time for the erection of the steel encasement by the proposed method is extremely short. Therefore, the cost of labor can be substantially reduced.

Direct fastening method has been widely used for connecting non-structural components. However, the application of this joining method to structural components has not been fully explored in the literature, not to mention the availability of relevant design guides. Lu et al. [14] conducted an experimental study and proposed design equations for screwed connections, but these are not applicable to connections joined by direct fastening owing to the different anchoring mechanisms. Lu et al. [15] further examined the behavior of connections joined by powder-actuated fastening in cold-formed steel sheeting at ambient and elevated temperatures. However, their study only involved joining very thin sheets with a single knurled fastener. Hence, to facilitate the application of direct fastening, a comprehensive study that takes the effects of more variables into consideration should be conducted.

In this study, the shear behavior of connections secured by direct fastening are experimentally investigated and design expressions are proposed. Factors such as type, number and arrangement of the fasteners, thickness of the base steel plate and fastener spacing have been considered. Furthermore, four columns divided into two groups by two sets of axial load ratios (ALRs) i.e. 0.16 and 0.30 are tested to validate the effectiveness of the steel encasement connected by direct fastening in improving the flexural strength and deformability of small rectangular RC columns which are commonly found in low-rise RC buildings.

2. Connection tests

The connections between the steel plates and angles in RC columns that are strengthened by using a steel encasement system with direct fastening connections should resist shear load incurred from the lateral load. The load transfer process of these connections is similar to single lap joints subjected to tensile force at the ends. Therefore, single lap joints shown in Fig.2 were designed and tested to study the failure process of the connections. In total, 100 specimens were fabricated for experiment. To clearly differentiate the different connection samples, they were labeled. Take for example, S275-A-4-5-6(30)-L: S275 denotes that the nominal yield strength of the steel material is 275 MPa; 'A' denotes that the sample is tested at room temperature; '4' represents the nominal diameter of the fastener; '5' signifies the thickness of the base steel plate; '6(30)' means that there are 6 fasteners with a fastener spacing of 30 mm; and 'L' means that the arrangement of these 6 fasteners is parallel to the load direction or Type 5 as illustrated in Fig.2. The thickness, width and length of connected steel plate are 3 mm, 100 mm and 200 mm. Thickness of the base steel plate varies from 3 mm to 6 mm while the width and length are same with those of connected steel plate.

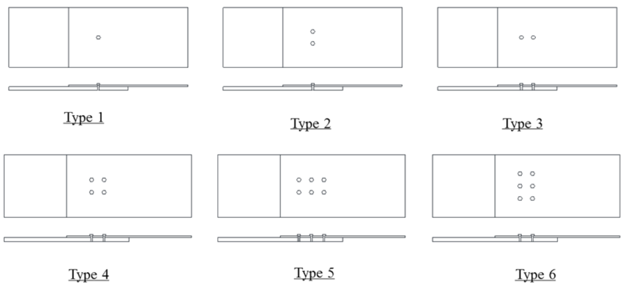


Fig. 2 – Configurations of tested specimens

Fig. 3 – Test setup

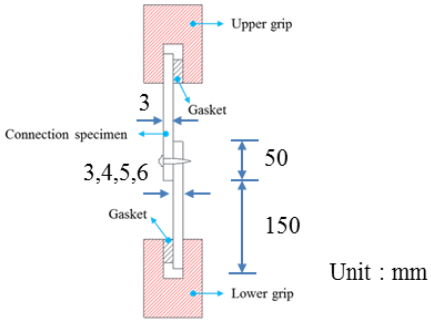


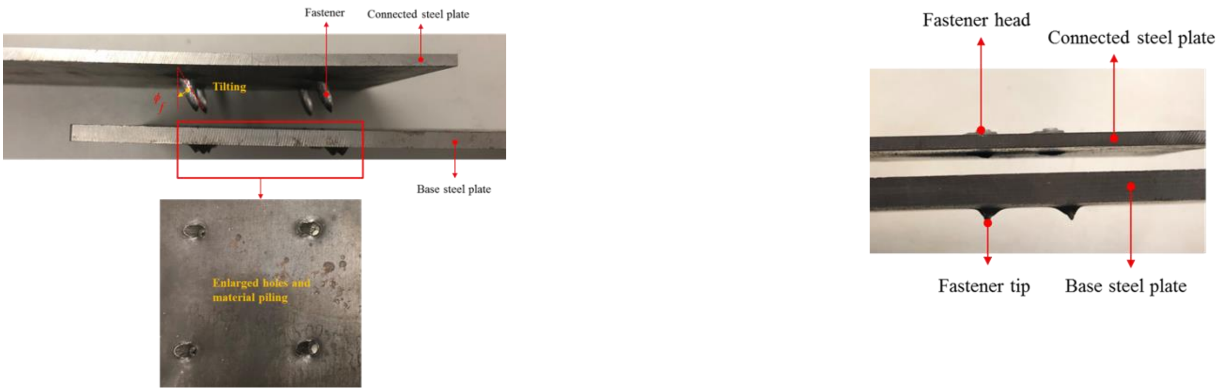
Fig. 4 – Assembly diagram of connection sample

2.1 Test setup and procedure

The MTS 810 material testing system (Fig.3) which has a maximum capacity of 250 kN was used to test the properties of the connection samples at an ambient temperature. Two gaskets were inserted between the upper and lower grips to allow the single shear of the connection samples (Fig.4). The rate of loading was 0.5 mm/min. The testing for each connection sample was terminated when the bearing resistance was reduced to less than 20% of the maximum bearing resistance to investigate its post-peak behavior.

2.2 Failure modes and failure behavior

Two kinds of failure modes - bearing failure and shear fracture failure, are observed in the tests. The typical failure modes are shown in Fig.5. Bearing failure results in enlarged fastener holes due to large plastic deformation and the bulging of the material around the fastener holes, which is shown in Fig.5(a). The bearing resistance versus displacement (BRVD) curves of the connections are plotted in Fig.6. Here, the BRVD curve for S275-A-4-4-1 is used as an example to explain the bearing failure process of this connection. In the initial loading stage, the steel plate is in an elastic region and the resistance force increases linearly with the displacement. When the steel material begins to yield, the load increases nonlinearly with the displacement, after which the bearing resistance remains approximately constant. During this stage, the holes on the steel plates are obviously enlarged and the fasteners are no longer normal to the steel plate. As a result, a pull-out force is carried out on the fasteners. The connection cannot take any force when the pull-out force exceeds the friction force between the fasteners and steel plates. Hence, the connection strength abruptly drops to zero, which is a significant shortcoming. Fig.5(a) shows the final rotation of the fasteners when the fasteners pulled out from base steel plate. The pull-out process does not occur at the same time for connections that are joined with more than one fastener, but sequentially during bearing failure. Therefore, there are inflection points on the BRVD curves when there is more than one fastener in the connection; see Fig.6.



(a) (b)

Fig. 5 – Failure modes: (a) bearing failure (b) shear fracture of fastener

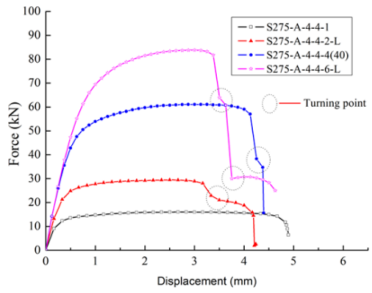


Fig. 6 – Shear load versus displacement of connections

2.3 Proposed equation for bearing strength

As stated above, the connections in this strengthening method are expected to resist shear force. The shear capacity of this connection should be quantified to determine the shear capacity of the strengthened RC column. Based on the equation in [16], a new equation is proposed, in which effect of protuberance and knurling is separated from the bearing factor.

$$F_b = \psi_{fp}\psi_{fk}\alpha_{br}d_n t_p f_{pu} \tag{1}$$

where α_{br} denotes the bearing factor; d_n represents the nominal diameter of the fastener; t_p is the thickness of the connected plate; f_{pu} depicts the ultimate strength of the steel material; ψ_{fp} is a factor for the effect of protuberance and it is equal to 1.0 for connections joined with pre-drilled holes on the connected plates while it is 1.35 for connections without pre-drilled holes on the connected plates; ψ_{fk} is included for the effect of knurling and it is 1.0 and 1.17 for fasteners that are not knurled and knurled, respectively.

The normalized peak load which is normalized by using the predicted peak load is presented in Fig.7. The average normalized peak load is 1.04 with a coefficient of variation (CV) of 0.11 and almost all of the normalized peak loads fall within the range of 0.8 to 1.2. Hence, the maximum bearing resistance can be accurately predicted by using the proposed equation with small discrepancies.

Bearing failure is the preferred failure mode in strengthening the connection due to its superior ductile behavior. To ensure that bearing failure occurs prior to fastener shear fracture, the following relationship needs to be satisfied with the exclusion of partial safety factors:

$$F_b < F_{fs} \tag{2}$$

where F_{fs} is the shear resistance of the fastener, which can be determined through tests.

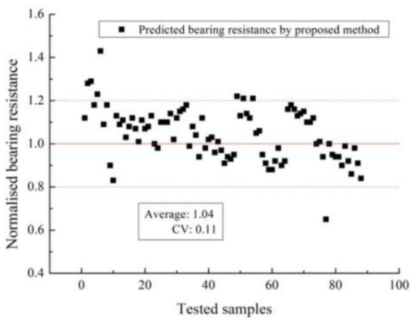


Fig. 7 – Comparison between predicted and measured peak loads

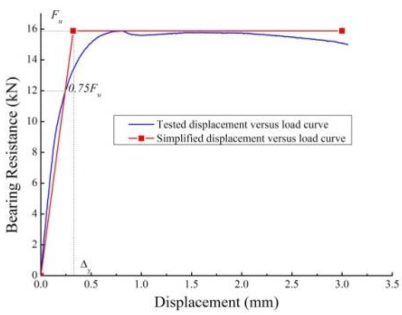


Fig. 8 – Simplified bilinear curve

2.4 Proposed equation of effective stiffness

The finite element method (FEM) is an effective numerical tool for analyzing composite structures. This approach can be used to simulate the structural behavior of strengthened columns by using steel encasement with direct fastening connections. To accurately simulate the structural behavior, the non-linear response of the connections secured with direct fastenings should be modeled. In this study, the non-linear bolt resistance-deformation curve is idealized as a bilinear curve as shown in Fig.8. This bilinear curve is governed by the bearing capacity and initial effective stiffness. The former has been presented in the previous section while the latter is presented as follows:

$$K_{ef} = \frac{\psi_{fn}\psi_{ef}E_s t_p d_n}{l_c} \tag{3}$$

where E_s is the elastic modulus of the plates; ψ_{ef} is the effective stiffness (ES) factor which is calibrated by the tested samples; ψ_{fn} represents the factor that depends on the number of fasteners (FN); l_c depicts the characteristic length in which deformation happens.

Owing to the fact the characteristic length is difficult to define, the effect of this parameter can be incorporated into the effective stiffness factor. Under this consideration, the effective stiffness can be rewritten as:

$$K_{ef} = \psi_{fn}\psi'_{ef}E_s t_p d_n \tag{4}$$

where ψ'_{ef} is the modified effective stiffness (MES) factor, which incorporates the effect of character length.

In total, 11 samples were used to determine the MES factor. The results are shown in Fig.9. A value of 0.017 is adopted for the MES factor. The relationship between the FN factor and number of fasteners is shown in Fig.10. It can be seen that the relationship is not exactly linear due to the group reduction effect. The average value is used for the FN factor for the number of fasteners in the test, and the FN factors are given by:

$$\psi_{fn} = \begin{cases} 1 & n_n = 1 \\ 1.4 & n_n = 2 \\ 1.9 & n_n = 4 \\ 2.1 & n_n = 6 \end{cases} \tag{4}$$

where n_n represents the number of fasteners. The FN factor for other quantities of fasteners can be determined by using linear interpolation and extrapolation.

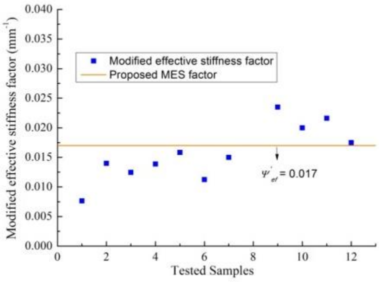


Fig. 9 – Modified effective stiffness factor

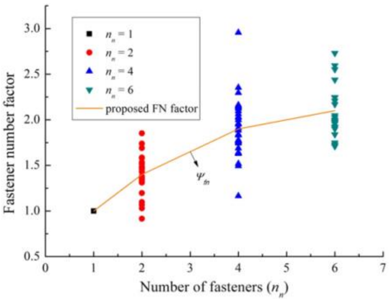


Fig. 10 – Factor depending on number of fasteners

3. Column tests

The column prototype was designed based on typical low-rise RC buildings. Owing to the restriction of the testing setup, the cross-section dimensions of the original column was scaled down to approximately one-third; a column with the height of 640 mm and cross-section dimensions of 150 mm × 150 mm is adopted in this test. Deformed steel rebars of 10 mm diameter and round bars of 6 mm diameter are used for longitudinal reinforcement and stirrup respectively. The target concrete strength is 30 MPa. Besides, the S275 grade steel plates with thickness of 4mm are used for steel encasement. The size of connected steel angle in steel encasement is 60 mm × 60 mm × 5 mm. In this proposed strengthening method, four steel plates with two ends bolted to the top and bottom of concrete blocks are directly affixed to an RC column to share the axial load and the lateral load. Steel angles are connected to the middle part of the adjoining steel plates by direct fastening to provide passive confinement to the concrete core and limit the slenderness ratio of the steel plates which is a key factor for maintaining the plate capacity. In this study, the stress lagging effect between the concrete core and external steel plates is neglected as the post-compressed load application method [17] can effectively mitigate this effect. To confirm the effectiveness of this proposed strengthening scheme, four columns including two control columns and two strengthened columns were constructed and then tested; see Fig.11. Furthermore, two sets of ALRs, i.e. 0.17 and 0.3 were examined to investigate the ALR effect on the column under the reversed cyclic lateral load and constant axial load.

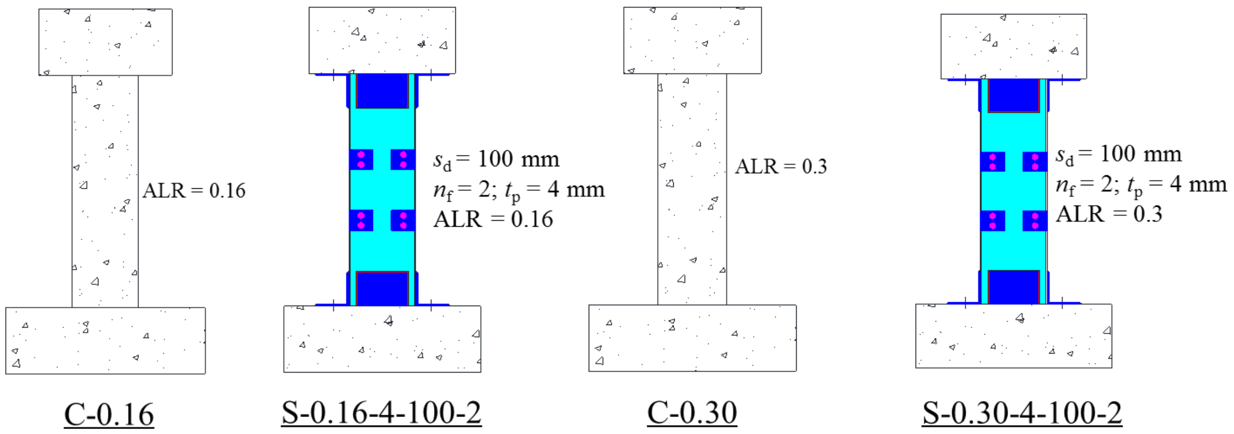


Fig. 11 – Configuration of tested samples

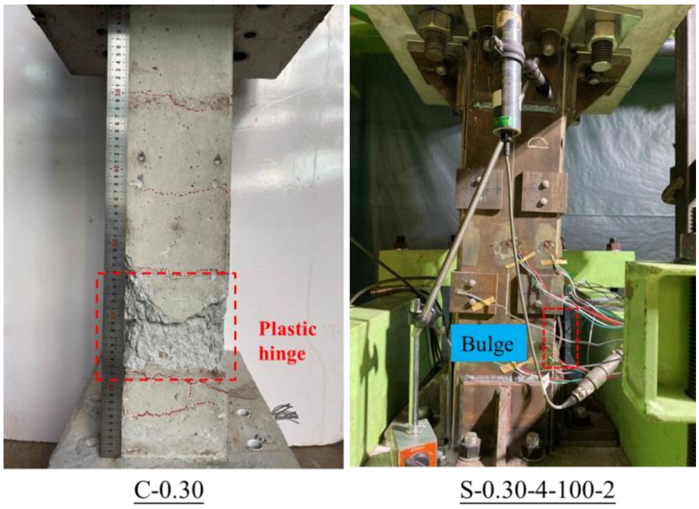


Fig. 12 – Failure modes

Fig.12 shows the failure modes of the tested samples. All the four samples were found to experience flexural failure, as evidenced from the cracks pattern at the base, without obvious diagonal shear cracks. Bulging of steel plates at the compressive side was observed when the peak lateral load is achieved. The connections play an important role in this strengthening method and they ensure stable and robust during the whole loading process.

The axial load is accordingly adjusted to realize the constant axial load condition because the axial actuator is hinged, which incur the minor saw teeth on the hysteresis curves shown in Fig.13. Prior reaching the peak lateral load, the two repeated hysteresis loops at each drift ratio almost overlap with each other. After the peak lateral load, the second hysteresis loops cannot match with the first hysteresis loop, where both stiffness and the strength decline. By incorporating the proposed strengthening method, both the strength and the deformability were greatly improved. Besides, the pinching effect is relieved in the strengthened samples.

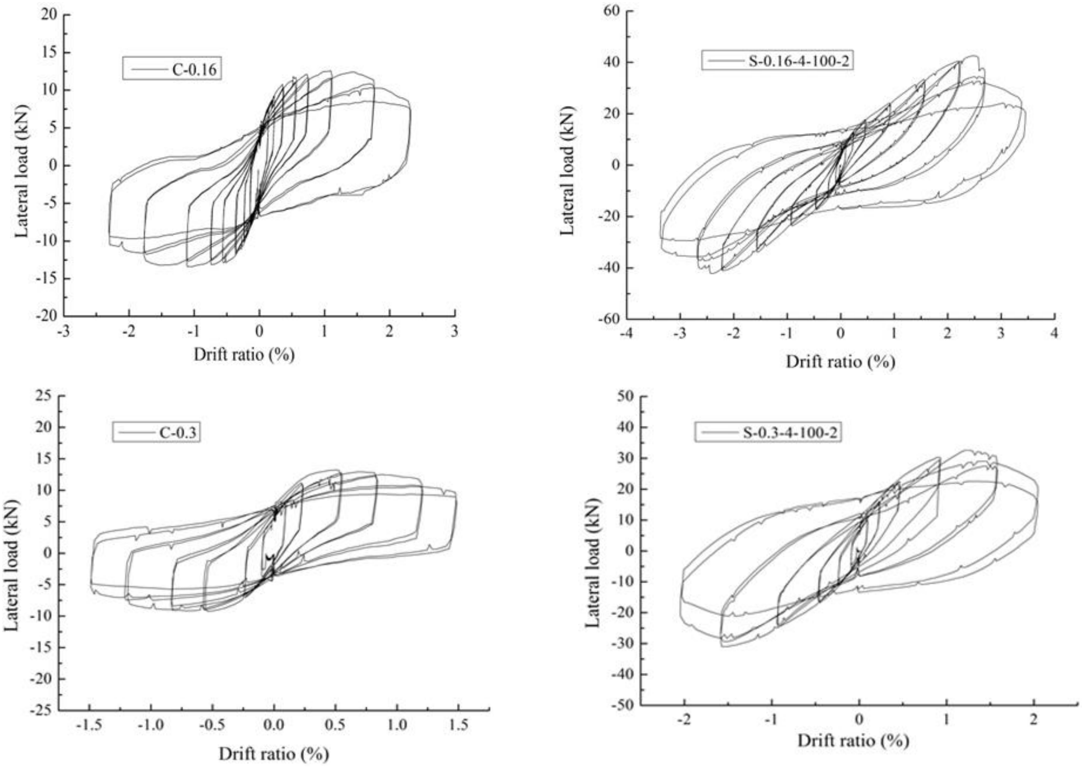


Fig. 13 – The hysteresis curves

To illustrate the effectiveness of the developed strengthening method under the effect of ALR, the envelope curves of first hysteresis loop at each drift ratio were extracted (see Fig.14) and the average maximum lateral load in two opposite loading direction are listed in Table 1. The ultimate drift ratio when the lateral load declines to 80% maximum lateral load is also summarized in Table 1. For S-0.16-4-100-2 and S-0.30-4-100-2, the shear strength increases by 226% and 184%, respectively and the ultimate drift ratio increases by 45% and 50%, respectively when compared to the control columns, i.e. C-0.16 and C-0.30. Besides, the ultimate drift of S-0.3-4-100-2 with ALR of 0.3 is comparable to that of the C-0.16 with ALR of 0.16. This indicates the use of the developed steel encasement significantly improve the shear strength and deformability of strengthened columns. Furthermore, the results show that the higher ALR produces detrimental effect on the strength and deformability of both the control columns and strengthened columns in this study.

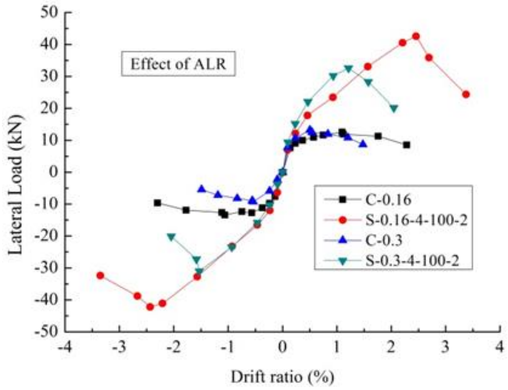


Fig. 14 – The envelope curves

Table 1 – Maximum lateral loads and ultimate drift ratios

Items	C-0.16	S-0.16-4-100-2	C-0.30	S-0.30-4-100-2
Maximum lateral loads (kN)	13.0	42.4	11.2	31.8
Ultimate drift ratios (%)	2.0	2.9	1.2	1.8

4. Conclusions

In this paper, a new seismic shear strengthening method for RC columns has been developed, in which the novel connection method plays a critical role. Therefore, comprehensive connection test was carried out to verify the robustness of connections joined by the direct fastening method. Following that, the column test was conducted to examine the effective of this strengthening method in improving the lateral strength and deformability. Some conclusions are drawn based on the work in this study.

(1) An empirical equation has been proposed to predict the shear resistant capacity of connections joined by direct fastening associated with bearing failure. Moreover, an equation for predicting the effective stiffness of the connection is proposed, in which the group effect is incorporated. Together with the empirical equation for predicting the shear resistant capacity of connections, a simplified bilinear curve can be determined for simulating the behavior of the connections jointed by direct fastening.

(2) The connections have demonstrated to be stable and robust during the whole loading process which allows the use of steel encasement to improve the shear strength and deformability of RC columns. Furthermore, it is found that high ALR is detrimental to the shear resistant capacity and deformability of both the control and strengthened columns.

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