Seismic Performance of a New Type Steel-Concrete Composite Shear Wall

Qilin Zhang¹, Yanan Huang¹, Guojun Xu², Lu Jiang²

¹Tongji Univ, Coll Civil Engn, 1239 Siping Rd, Shanghai 200092, Peoples R China
²Jinggong Green Building System Crop. Limited, 999 Li’an Rd, Shanghai 200092, Peoples R China

Iyananhuang@tongji.edu.cn

Abstract. Using steel-concrete composite section is an efficient method for improving the behaviour of shear wall under seismic action. In consideration of the complex configuration of traditional composite shear wall and the time-consuming constructing process, based on the existing research, partially encased composite section was introduced into shear wall system. The partially encased composite shear wall (PECSW) is composed of a steel bone, some horizontal links and a group of concrete columns. The steel bone is a steel web, welded with vertical ribs and flanges in both side at certain interval. The horizontal links connect the flanges (or vertical ribs) and vertical ribs. Concrete is poured on both sides of steel web, between flanges (or vertical ribs) and vertical ribs, formulating several independent long concrete columns. The PECSW can be bulk prefabricated in the factory and transported to the site to install. Because of no vertical rebar in PECSWs, the PECSW structure, which consists of PECSWs and other precast concrete structural elements, can be assembled quickly. This paper reports an experimental study on the seismic behaviour of the PECSW under cyclic lateral loading. Two full-scale single-bay, single-story specimens were constructed. The bearing capacity, energy consumption, stiffness and other performance data had been discussed based on the results of experiment. To explore the possibility of simplifying configuration of PECSW, the web of one specimen was welded with stud shear connectors on both side while the web of another specimen wasn’t. The test results show that the PEC shear wall has a good seismic behaviour. Both specimens followed bending failure mode. The concrete columns offered the vertical bearing capacity as well as the flexural bearing capacity. The concrete and the horizontal links between flanges and ribs provided effective support against local buckling of the flanges, once broken occurred between the links and the flanges, the flanges buckled severely, and the bearing capacity of PECSW fell accordingly. The initial stiffness, yield drift, peak point bearing capacity, accumulated energy dissipation of the PECSW specimens with and without stud shear connectors was basically identical. Then, parametric studies were conducted through numerical simulation so that the contribution of links and the stud shear connectors can be evaluated. According to the research above, the concrete part in the PECSW can postpone local buckling of steel sheet. The horizontal links in the PECSW support interaction between steel and concrete, provide concrete anchorage to steel bone. In general, the PECSW has commendable seismic behaviours and deserve further study.

1. Introduction
The steel-concrete (SC) composite shear wall is an excellent form of shear wall system. Because the steel bone embedded in concrete, on the one hand, it overcomes the shortcomings of the steel plate shear wall itself cannot bear the vertical load; On the other hand, its energy dissipation and ductility
under horizontal action are better than those of the reinforced concrete shear wall. Because there are reinforced rebar as well as steel bone in it, the construction of structures with SC composite shear walls is complicated and time-consuming. Proposing a composite shear wall system of simple configuration to save construction time is necessary.

The PEC column was presented firstly as a good solution for fire resistance problem of steel members because of the presence of concrete [1]. Whereafter, behaviour researches of it conducted by Ballio et al. [2] confirmed its superior mechanical performance. This PEC column section (Typical European section, shown in Figure 1(a)) has been involved in Eurocode4 [3]. Then, a modified form of PEC columns were designed [4] by researchers of Imperial College. In this section (Modified Section, shown in Figure 1(b)), transverse links, which were welded between flanges, were alternately placed with stirrups (welded to web) along the longitudinal direction in the plastic hinge area. After experimental and numerical researches [5], conclusion had been given that the transverse links played an important role in improving the performance of PEC column. Subsequently, a new type of PEC columns (New Section, shown in Figure 1(c)), without any stirrups or longitudinal rebar, was proposed by Elnashai and his co-researchers [6-8]. This section consisted of H shape of thin flange, concrete encased by flanges and web, and two groups of transverse links welded between flanges at certain interval. Quasi-static tests and pseudo-dynamic tests have been conducted, analytical model was proposed and parametric investigations based on this model had been done. Chicoine et al. [9-11], Bouchereau and Toupin [12], Prickett and Driver [13], Begum et al. [14], Youzhen Fang [15-17], Yiyi Chen et al. [18] carried out a series of theoretical and experimental studies on the PEC column with new section, and proved that this kind of column guaranteed good seismic performance with simplified configuration, which is worthy of application in actual engineering. Based on some of the studies mentioned above, design equations for PEC columns under axial load, pure moment, and combined axial load and bending moment have been provided in the Canadian steel design standard, CSA S16-14 [19].

By reason of good performance of new section PEC columns, Driver and his research group [20-23] proposed a new type of steel plate shear wall with PEC column as the end columns, and carried out a series of experimental research on this system. Experimental research and numerical simulation of this shear wall system also have been conducted by Zhanzhong Yin et al. [24-26], Yu, Jin-guang et al. [27]. Results indicated that the steel wall with PEC end column system has better stiffness, bearing capacity, ductility and energy dissipation performance than the steel wall with H-shaped steel end column. However, this system still cannot improve the shortcomings of low structural rigidity and inability to withstand vertical loads.

![Figure 1. Section of partially encased composite column.](image_url)
Based on the conditions described above, the partially encased composite shear wall (PECSW) system was proposed in this paper. Steel bone in this wall is composed of steel web, flanges and vertical ribs, which form a series of continuous I-shaped cross sections together. A group of sheet steel, with a width of 30mm and a thickness of 3mm, were welded between flanges (or vertical ribs) and vertical ribs along the vertical direction at certain interval. Concrete was poured between the flanges (or ribs) and the ribs, and existed in PECSWs in the form of slender columns which were separated by web and ribs. The configuration of the PECSW is shown in Figure 2. Elements of PECSW structures can be bulk prefabricated in the factory and transported to the site to install. Since no longitudinal bar is contained and the flanges are exposed in PECSWs, both longitudinal and horizontal splicing is simplified, so that the construction period of PECSW structures can be effectively shortened. There will be good economic benefits when the PECSW is used in mid-to high-rise shear wall structures.

This paper presents the results of preliminary experimental and numerical research on the seismic performance of PECSW under compression and bending. This study can provide the reliable experimental results facilitating the actual application of PECSW.

2. Experimental programme

2.1. Description of specimens

2 full-scale single-bay specimens were constructed for pseudo-static test, which were numbered as S1, S2, respectively. The geometric dimensions and details of the specimens are summarized in Table 1, and shown in Figure 3.
Table 1. Details of the PECSW specimens.

| Number | Thickness / mm | Strength | stud shear connectors | Design axial compression ratio $\eta$ | Axial compressive load / kN |
|--------|----------------|----------|-----------------------|--------------------------------------|-----------------------------|
|        | Flanges | Side ribs | Middle rib | web | concrete | steel bone | No | 0.4 | 2944 |
| S1     | 10      | 10        | 6          | 8   | C35       | Q235       | Yes | 0.4 | 2944 |

*aThe calculating method of $\eta$ can be found in China standard [28][29].

2.2. Materials

The property of steel sheet and concrete used in the experiment was tested according to standard GB/T 228.1–2010 [30] and GB/T50081-2002[31], respectively. The mean value of strength and young's modulus based on the test data are shown in Table 2.

Table 2. Mean value of strength and young’s modulus of material in specimens.

| Steel | Concrete |
|-------|----------|
| Yield strength (MPa) | Ultimate strength(MPa) | Cubic compressive strength (MPa) | Young’s modulus E0 (N/mm²) |
| 328   | 482      | 34.14    | 31075 |

Figure 3. Configuration of the specimens.
2.3. Test devices and procedures
The specimen was connected to the patand by anchor bolts. Vertical load (The value can be found in Table 1), applied by a 100t MST vertical actuator, is converted by the rigid distribution beam into a uniform load applied to the top surface of the loading beam. After all the vertical load was loaded, it remained unchanged throughout the test. The horizontal load was applied to specimen by the 150t MTS horizontal actuator, which was connected to the loading beam of specimen by connecting set. Horizontal loading in this test is controlled by displacement [32]. Test device is shown in Figure 4.

![Test loading device](image)

**Figure 4.** Test loading device.

![Cavity identification](image)

**Figure 5.** Cavity identification.

**Table 3.** The main experimental phenomena.

| Loading Step | Phenomena |
|--------------|-----------|
| The early step | Before yielding, a small amount of horizontal cracks were observed at the bottom of the side cavity concrete of the two specimens. |
| 2 $\Delta y$ | Horizontal cracks and slight bulging appeared at the bottom of the side cavity concrete. |
| 3 $\Delta y$ | The concrete covering peeled off because of the aggravation of concrete bulge. |
| 4 $\Delta y$ | Longitudinal cracks came into being in the lower part of the side cavity concrete. Lower part of flanges buckled outward. Weld between flange and transverse links in the plastic zone of S1 fractured, concrete within this region crushed and dropped out of wall. |
| 5 $\Delta y$ | The bearing capacity of S1 declined to 85% of the peak point bearing capacity. Longitudinal crack arose in the lower part of the second cavity concrete of S2. Weld between flange and transverse links in the plastic zone of S2 broke. Flanges near the fractured links buckled outward obviously. Concrete nearby the fractured links crushed seriously and dropped out of wall. The bearing capacity of S2 declined too. |
| 6.5 $\Delta y$ | The bearing capacity of S2 declined to 85% of the peak point bearing capacity. |

3. Experimental results

3.1. Experimental phenomena and failure mode
Before the test, no obvious cracks were found on the wall surface of the two specimens. During the test, both of the specimens were destroyed by bending and compression, the main phenomena listed in Table 3, and the final failure mode of specimens is showed in Figure 6 and Figure 7. For convenience of description, the definition of each cavity (enveloped by web and flanges and/or ribs) on PECSW
specimens is shown in Figure 5. In the table 3, if the experimental phenomenon didn’t indicate in which specimen it appeared, it means that the phenomenon occurred in both specimens.

![Figure 6. Final damage mode of S1.](image)

3.2. Experimental data analysis

3.2.1. Hysteretic loops and skeleton curves. The horizontal load (P) - displacement (Δ) hysteretic loops and the skeleton curves are not only a comprehensive reflection of seismic performance, but also the main basis of the analysis of elastoplastic seismic dynamic response. The hysteretic loops at the horizontal loading point and the skeleton curves of specimens is shown in Figure 8.
As shown in the Figure 8, the hysteresis curves and the skeleton curves of the two specimens were basically coincident in the elastic phase and elastoplastic phase, which means the initial value and degradation of stiffness of the two specimens were similar. In the elastic phase, the load-displacement curve developed linearly and the envelope area of hysteresis loop was small. When the flanges began to yield (detected by strain gauge located on the bottom of flanges), the slope of load-displacement curve slowed down, the hysteretic loops were gradually turned to be spindle-shaped, depending on PECSW’s good ability to maintain stiffness. When the transverse links were disconnected from flanges, the interaction between steel and concrete vanished accordingly, concrete was squeezed out of the wall, and the buckling of the steel flange and web was more serious than before. As a result, the bearing capacity began to drop. Finally, due to the serious local buckling of steel flanges and web and the crushing of concrete, the specimens were damaged. As different the welding flaw was, fracture of welds between transverse links and flanges occurred in different loading step in the tests of the two specimens, which resulted in different performance of descending stage of hysteresis curve and skeleton curve.

In order to evaluate and compare the performance of PECSWs with and without stud shear connectors, bearing capacity, deformation capacity, stiffness degradation and energy dissipation were discussed as follows.

3.2.2. Bearing capacity and deformation capacity. The nominal yield point of the specimen can be obtained from the skeleton curves by the energy equivalent method [33], the ultimate point of the specimen is the point on the skeleton curve where the load dropped to 85% of the peak load.

The deformation capacity can be evaluated by the ductility coefficient and the drift. The ductility coefficient was designated as the ratio of maximum deformation to yield point deformation [34]. The drift is the ratio of deformed value to the height of specimen. The corresponding values of the two specimens are shown in Table 4.

Table 4. The comparison of specimens’ corresponding values.

| Specimen | Loading direction | Yield point $P_y/kN$ | Drift | Peak point $P_u/kN$ | Drift | Ultimate point $P_{ult}/kN$ | Drift | Ductility coefficient |
|----------|------------------|---------------------|-------|---------------------|-------|-----------------------------|-------|----------------------|
| S1       | Push             | 701.86              | 0.49% | 1019.70             | 1.01% | 1.73%                       | 3.54  |                     |
|          | pull             | -749.24             | 0.48% | -1018.80            | 0.97% | 1.54%                       | 3.21  |                     |
| Average  |                  | 725.55              | 0.49% | 1019.25             | 0.99% | 1.64%                       | 3.37  |                     |
| S2       | Push             | 781.81              | 0.58% | 1067.90             | 1.59% | 2.08%                       | 3.57  |                     |
|          | pull             | -728.48             | 0.44% | -1027.20            | 1.25% | 1.92%                       | 4.40  |                     |
| Average  |                  | 755.15              | 0.51% | 1047.55             | 1.42% | 2.00%                       | 3.98  |                     |

As can be seen from Table 4, the mean yielded load of S1 was 725.55kN, which was 4.08% lower than that of S2. The peak load of S1—1019.25kN—was 2.78% lower than 1047.55kN of S2. The ratio of peak drift to yield drift of the two specimens was greater than 2, indicating that the specimens would undergo a long-term deformation process before reaching the peak load. S1 yielded with a drift.
of 0.49%, and reached the peak load with a drift of 0.99%, both earlier than that of S2. Its ultimate drift was 1.64%, which was 22.03% lower than the 2.00% of the S2. The two ductility coefficients of the specimen were both greater than 3, which indicated there would be obvious signs before specimens were damaged. Its ductility coefficient of 3.37 was 18.10% lower than the 3.98 of S2. Table 4 showed that the bearing and deformation capacity of S2 exhibited higher than that of S1 because of the later fracture of weld between links and flanges, but the two specimens were not much different.

3.2.3. Stiffness degradation. The stiffness degradation of the specimen is described by secant stiffness under cyclic loading[35]. The secant stiffness is the ratio of the average load of the point with the largest displacement to the average maximum displacement of the three circles under the same loading step. The change trend of tangent stiffness of each specimen with loading is shown in Figure 9.

![Figure 9. Stiffness degradation of the specimens.](image)

![Figure 10. Energy dissipation of the specimens.](image)

As shown in the Figure 9, the initial stiffness of S2 was slightly greater than that of S1, but it quickly decreased to the same value as S1. Then, the secant stiffness of the two specimens remained similar until weld of S1 between links and flanges fractured. In the decline stage of bearing capacity, the secant stiffness degradation rate of S2 was slightly slower than that of S1.

3.2.4. Energy dissipation. The energy dissipation capacity of the specimens, measured by the area enclosed by the hysteresis loop in the load-displacement hysteretic curve, is one of the important indicators to evaluate seismic performances of the specimen. Figure 10 shows the evolution pattern of energy consumption (from the first cycle of each loading step) with loading.

It can be seen from Figure 10 that the energy dissipation of each specimen increased steadily and significantly with the increase of the displacement. The energy dissipation of S1 was slightly greater than that of S2 under the same displacement, but the energy dissipation of the two specimens was not much different.

In short, in the quasi-static loading experiment, PECSW specimens had good performance in terms of bearing capacity, deformation, maintaining stiffness and energy dissipation. Although there were some differences in the test results of S1 and S2, the differences are very small. There were two main reasons for the differences: one was whether specimen had stud shear connectors, and the other was the loading step when fracture of welds between links and flanges occurred. The influences of reasons above will be evaluated in the next section.

4. Finite element model

4.1. Description of FE models

Since coupling influence of stud shear connectors and broken links to specimens has been observed in test, numerical analyses were performed to further research by the finite-element software ABAQUS. Solid element C3D8R was used to simulate other parts of the PECSW specimens except for the foundation reinforcement (truss element T3D2). In order to ensure the calculation accuracy, the mesh at the lower part of the wall was densified.

The concrete of specimen with stud shear connectors interacted with the steel web by “tie” constraint and “contact” interaction with steel ribs and flanges, while the concrete of specimen without stud shear connectors interacted with all steel parts by “contact” interaction. The properties of contact interaction consist of “hard contact” in normal direction and “Penalty Friction” with a friction coefficient of 0.4 in tangential direction. Transverse links interacted with concrete by “embedded”
constraint. The disappearance of the concrete confinement caused by the fracture of the welds between transverse links and flanges can significantly change the mechanical properties of the concrete, and this change cannot be achieved precisely in ABAQUS, so the fracture of welds was not been considered in FE models.

In order to accurately simulate the actual working conditions of the specimen, the boundary conditions of the FE model should be consistent with the experimental conditions. The concrete base as well as the reinforcement ("embedded" in concrete base) of it were established in FE model, which was connected to the ground (simulated by rigid body, the reference point of which was fixed) by bolts. The steel bone of wall was connected with concrete base through a steel beam "embedded" in the latter. The concrete part of wall was connected with the top surface of concrete base through "tie" constraint. The loading beam located on the top of wall was replaced by kinematic coupling constraint. The axial load was applied to the top of the wall in the form of a uniform pressure, and the horizontal load was applied to the reference point (located on the middle of top surface of wall) in the form of displacement. The detailed information of FE model is shown in Figure 11.

4.2. Material modelling
The input parameters of material properties were determined based on the data obtained from the material tests. Combined hardening module was adopted to define the material properties of steel material. Concrete damaged plasticity (CDP) model was used for concrete material.

The tensile hardening law of concrete adopted the uniaxial tensile stress-strain relationship given in China standard [28]. In order to take the influence of the concrete confinement into account, the concrete is divided into confined area and unconfined area based on the idea of the Xianzhong Zhao [36]. Mander model [37] is used for the compression hardening law of concrete in the confined area, and the uniaxial compression stress-strain relationship given in China standard [28] is used for the compression hardening law of concrete in the unconfined area. The stress-strain relationships above were calculated and entered according to the format required by the CDP model in ABAQUS.

5. Numerical study
As shown in Figure 12, load-displacement curves obtained from FE models (FE) of S1 and S2 were compared with the corresponding curves of experimental results (EX), respectively. The curves of both FE models before the fracture of welds between transverse links and flanges occurred fit well with the experimental curves. However, after the break of welds (in experiments), the bearing capacity of FE model is significantly higher than the test result at every hysteresis loop. The main reasons is that:

1. The stress-strain relationship of concrete after ultimate compressive strain $\varepsilon_{cu}$ was not given in FE model, so when strain of concrete is larger than $\varepsilon_{cu}$, the corresponding stress was constant, which was inconsistent with reality.
2. Concrete spalled at the bottom of concrete column in side cavity cannot be simulated accurately in FE model, thus the degeneration of bearing capacity and stiffness based on it cannot be reflected in FE model.

3. After fracture of welds between transverse links and flanges occurred in experiments, bearing capacity of concrete slumped because of vanishment of concrete confinement, so that the lower part of concrete columns in side cavity was crushed and extruded from the wall. Whereafter, steel web and flanges nearby the crushed concrete buckled seriously due to the loss of lateral support provided by concrete. These phenomena above cannot be accurately simulated in the FE model.

Although the failure of PECSW specimens cannot be simulated accurately, the stiffness, bearing capacity, the strain distribution of FE model were similar to test results. The impact of the stud shear connectors on the seismic performance of PECSWs can be evaluated based on the comparison of the results obtained from the two FE models, and the influence of the disconnection of transverse links and flanges on the seismic performance of PECSWs can be studied by comparing the results of the finite element model and the corresponding test results of each specimen.

By comparing the test data and finite element results of S1 (Figure 12 (a)), as well as the two sets of data of S2 (Figure 12 (b)), it can be seen that the fracture of the welds between transverse links and flanges has a rapid and obvious influence on the bearing capacity of PECSW members. S1 fractured the welds on both sides during the loading step of 4 Δy, S2 fractured the welds on the left side during the loading step of 5 Δy, and the corresponding bearing capacities of these steps were significantly reduced compared with the previous step. Therefore, it suggested that the transvers links in the plastic hinge zone have a very important effect on the bearing capacity and seismic performance of the PECSW members. It is necessary to study the welding method of the weld between transverse links and the flanges to ensure the reliability of the connection.

By comparing the load-displacement curves obtained from the finite element models of S1 and S2 (Figure 12 (c)), it can be seen that the stiffness and load-bearing capacity of the specimen can be slightly improved by the use of the stud shear connectors. In PECSW members, the concrete column is wrapped by web, flanges and vertical ribs, and the transverse links acted just like stud shear connectors. So the relative slip between concrete and steel is so small that it is negligible. Therefore, in practical applications, the surface of the steel web of PECSW does not need to be welded with shear studs. For finite element simulation, the interaction between concrete surface and the steel web surface can be set as “tie” constraint, which can ensure calculation accuracy and save analysis time at the same time.

(a) Comparison of experiment result with FE simulation result of S1. (b) Comparison of experiment result with FE simulation result of S2. (c) FE result comparison of S1 with S2.

Figure 12. Comparison of hysteresis curves.

6. Conclusions
The PECSW structure is proposed in this paper. Based on the pseudo-static experiment and numerical analysis, the seismic performance of the PECSW member was researched, and the influence of the stud shear connectors located on steel web and transverse links on its performance had been discussed. Based on the results of tests and numerical analysis, the following conclusions were obtained:
1. The PECSW member has excellent performance in bearing capacity, deformation, energy consumption, stiffness, etc. because of the interaction of concrete and steel bone, and it is worthy of further research.

2. In PECSW members, with or without the shear connectors on steel web has a little effect on its mechanical performance which can be ignored.

3. The influence of transverse links in plastic hinge areas on the seismic performance of PECSW member is significant. Once disconnected form the flange, the bearing capacity will drop rapidly. Therefore, it is necessary to ensure the reliability of the connection between the transverse links and steel bone in PECSW members.

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