Cyclic loading test for precast concrete composite shear walls

Yubin Sheng1, Zijun Wang1,3, Yu Wang2, Handong Yang4*

1 Department of civil engineering, Nanjing Tech university, No 30, Puzhu Road(s), Nanjing, China.
2 Shanghai Construction Steel Structure (Jiangsu) Co, LTD, Nantong, China.
3 Jiangsu Tian Gong Building Science and Technology Group Co, LTD, Huaian, China.
4 Jinhu Construction Market Control Office, Huaian, China.
Email: 565868392@qq.com

Abstract. Shear wall systems as horizontal load resisting system are used widely in high-risen building. An innovative shear wall, named Precast Concrete (PC) composite shear wall with confined boundary elements, is proposed in this paper. The new-style structural wall is composed of PC panels, in-filled concrete and confined boundary members. Three full-scale specimens were subjected to cyclic lateral load under axial load. These structural characteristics were investigated and analysed including bearing capacity, hysteretic loops, skeleton curves, ductility, and energy dissipation under cyclic load. The specimens exhibited good ductility and underwent stable seismic behavior. Specially, the precast shear wall with joint motor show sufficient ductility, deformation capacity, stiffness and energy dissipation. The specimens experience the failure in the sequence of concrete crushing and the propagation fractures at the boundary of the wall. The experimental results indicate that the confined boundary elements have a great remarkable effect on the precast shear wall.

1. Introduction
Traditionally, the reinforced concrete shear walls have been widely used as the horizontal load resisting system in multi-story building, especially in the high-rise building. In concrete building, concrete shear walls carried with the confined boundary elements have been in use to resist seismic excitation [1]. In addition, the production of traditional concrete shear walls mainly focus on reinforced banding, steel welding and supporting formwork, which results in the amount of formwork consumption rising, the serious waste of construction materials and plenty of construction waste occurring. Thus, the production mode of shear walls cannot meet the sustainable development requirements including the high efficient utilization of resources, energy-saving and environmental production.

Recently, the Precast Concrete (PC) shear walls [2-6] have been applied to a number of buildings as a lateral-load resisting system and obtain satisfactory results including construction efficiency and economic advantage. Soudki [7-8] does research on horizontal connections for PC shear walls. As a result, the unbound rebars with sleeve connection exhibits sufficient ductility, deformation capacity and energy dissipation capacity. Unbound prestressed PC shear walls are conducted as a lateral-load resisting system to resist seismic effect. Unbound prestressed PC shear walls [9-15] combine the good characteristic equivalent to cast-in-place concrete shear wall. Which can be one of the alternative for new forms of structural system attributed to sufficient shear resistance and deformation capacity. However, using unbound tendon does not have a positive influence on the energy dissipation, which in turn leads to the severe decrease in energy dissipation capacity. A common solution to enhance the energy dissipation capacity is to make use of mild steel bars as a hysteretic energy dissipation device.
[12-13].

In the present study, a new-style shear wall, namely precast composite shear wall with confined boundary elements, is proposed. Two PC panels were connected by the scissor support, in order to enhance the integrity of the shear wall. Simultaneously, concrete is pour inside the cavity to act compositely with the PC panels. Therefore, the arrangement of the shear wall conduces to satisfying the requirement that the composite shear wall is expected to improve the seismic performance equivalent with the cast-in-place concrete shear wall while reducing the amount of formwork and the waste of construction material. Furthermore, in contrast to the most previous precast shear wall systems that serve as lateral-load resisting systems only, the precast composite shear wall is designed to bear both the lateral load and the vertical load. The seismic performance of the precast composite shear wall was evaluated in the field of hysteresis loops, ductility, stiffness, energy dissipation capacity and failure mode.

2. Experimental Program

2.1. Specimen Design

The test specimens are designed to simulate the structural walls of the high-rise buildings. Three full-scaled shear walls labeled from W-1 to W-3 were tested under pseudo-static cyclic loading to investigate the seismic behavior of the proposed shear walls. Figure 1 shows the dimensions and details of the specimens. The wall height \( h \) from the bottom to the loading point was 1850mm. The wall depth and thickness were 1400mm and 160mm, respectively. Thus, the aspect ratio of the walls, \( h/l \), was 1.32. The axial force ratios for all specimens are 0.2. The geometric detail of the precast composite walls is provided in Figure 1 and Table 1. The parameters studied in this research included the configuration of the wall, the presence of confined boundary member and loading history.

(a) Specimen W-1
Figure 1. Arrangement of reinforcement and dimensions for specimens.

Figure 1(a) shows the precast composite concrete Specimen W-1. The Specimen W-1 with a cross-section of 1400 mm×250 mm consists of both 1400 mm×50 mm PC panel and 1400 mm×40 mm PC panel at the two boundaries and 1400 mm×50 mm insulating layer attached to the PC panel with the thickness of 40 mm as given in Figure 1(a). Specimen W-1, Specimen W-2 and Specimen W-3 have identical properties except that the confined boundary members. The detail of other specimens can be illustrated in Figure 1(b) and Figure 1(c) respectively. The reinforced allocation and structure
measure of all specimens are satisfied with Earthquake Resistant Building (GB 50011-2010) [16] and Technical Specification For Concrete Structure Of Tall Building (JGJ 3-2010) [17].

| Type | Height of wall (mm) | Cross section (mm×mm) | Effective thickness (mm) | Axial load ratio | Axial force (kN) |
|------|--------------------|------------------------|-------------------------|------------------|------------------|
| W-1  | 1850               | 1400×250               | 160                     | 0.2              | 1173             |
| W-2  | 1850               | 1400×250               | 160                     | 0.2              | 1173             |
| W-3  | 1850               | 1400×250               | 160                     | 0.2              | 1173             |

2.2. Material Properties
The cast-in-place concrete and PC used in the specimens has a strength grade of C30 (normal cubic compressive strength $f_{cu,d}=30$ MPa, designed value of axial compressive strength $f_{c,d}=14.3$ MPa). Nine 150 mm size cubes was produced and cured for 28 days. The concrete compressive strength for the PC panels is 50.4 MPa. The cast-in-place concrete compressive strength for Specimens W-1, W-2 and W-3 is 33.2 MPa. Attention should be paid to the axial compressive strength of concrete $f_{c,t}$ is regarded as $0.76f_{cu,t}$ according to the Chinese Code for Design of Concrete Structures GB50010-2010 [18]. Three varieties of steel bars received from tensile bar, distributing bar and confine boundary members respectively were tested to determine the yield strength $f_y$ and ultimate tensile strength $f_u$.

| Type       | Strength rank | $d$ (mm) | $f_y$ (MPa) | $f_u$ (MPa) | $f_y/f_u$ | $E_s$ (MPa) |
|------------|---------------|----------|-------------|-------------|-----------|-------------|
| Steel bar  | HPB300        | 6        | 433.9       | 500.2       | 0.87      | $2.1\times10^7$ |
|            | HRB400        | 10       | 575.2       | 594.2       | 0.97      | $2.0\times10^5$ |
|            | HRB400        | 14       | 461.0       | 600.3       | 0.77      | $2.0\times10^5$ |

2.3. Test Setup and Loading Procedure
The specimens were tested by the strong functional loading device in Nanjing Tech University. The horizontal loading capacity of the device was 1400 kN. The general detail of the test setup is shown in Figure 2. All specimens were tested under constant vertical load and cyclically increasing horizontal load. The axial force was taken into the specimen initially by vertical hoisting jack and maintained constantly during the test. Afterwards, cyclic lateral force was applied by the horizontal hydraulic actuators fixed horizontally to the reaction wall.

The lateral cyclic loading history was generally relied on the testing protocol specified in the Chinese Specification of Testing Methods for Earthquake Resistant Building (JGJ 101-2015) [19]. The lateral force was controlled by load, before the specimen yielded. In the load-controlled stage, lateral force was carried out in three levels, which accorded with 1/3, 2/3, and 1 of the predicted yield load of the specimen. The lateral force was converted to displacement-controlled after the specimen yielded. For all specimens, the displacement increment in accord with $D_y$ and three cycles were repeated at every displacement level as illustrated in Figure 3. In every loading cycle, a push (from the left to the right in Figure 2) was performed first, immediately followed by a pull (from the right to the left). The test was terminated when the specimen was unable to bear the axial load or the lateral force decreased below 85% of the maximum load.
Figure 2. Experimental set-up and layout of LVDTS.

Figure 3. Loading history.
2.4. Instrumentation
The vertical and lateral loads could be recorded by using the load cells attached to the hydraulic actuators. Nine linear variable differential transformers (LVDTs) were applied to measure the deformation of the wall. LVDT H-1 was installed at the top as high as the load point to record the lateral displacement of the wall. LVDT H-2, H-3, and H-4 were distributed along the height of the specimen to monitor the lateral displacement. LVDT H-5, V-6, and V-7 were placed on the foundation beam. The horizontal slippage of the foundation beams was measured by LVDT H-5. Another two were used to record the torsion which might be induced by the possible uplift of the foundation beam. LVDT H-8 and H-9 installed along the diagonal line of the wall were employed to monitor the shear deformation of the specimens. The overall arrangement of displacement transducers can be observed from Figure 2.

3. Test Results

3.1. General Behavior

3.1.1. Specimen W-1

When the lateral load was acted on the Specimen W-1, no obvious crack or other sign could be observed at the early stage. The initial horizontal crack was occurred from the tensile region of the PC panel and spread to the central region of the wall. As the lateral load increasing, a more continuous new crack emerged from other location. The lateral displacement of 6.4 mm acted as the yield displacement of the specimen as well as the corresponding lateral force was 372.4 kN in the test. During this stage, new cracks could be observed from the seam between the notch of the PC panels and the post-poured concrete strip, and continued to propagate along the horizontal seam quickly. When the lateral displacement arrived at 19.9 mm, the crack proceeded to aggravating seriously as well as concrete crumbling began to occur at the bottom of the boundary of the specimen. Until the displacement reaching 25 mm, no sever crack took place in the PC panels and concrete crushing at the bottom of the specimen proceeded gradually as shown in Figure 4. The lateral force decreased to below 85% of the peak load of the specimen, and the test stopped. The failure condition is illustrated in Figure 5.

3.1.2. Specimen W-2

Specimen W-2 showed a stable hysteresis loop at the early stage. The horizontal load increased linearly with an increase in the lateral displacement. When the lateral load was up to 220 kN, the horizontal crack took place from the bottom of the wall. As the increase in the horizontal force, the crack continued to grow up as well as the new crack occurred. The lateral displacement of 7.3 mm was defined as the yield displacement and the corresponding horizontal load 456.2 kN. During the yield stage, attention should be paid to that the specimen exhibited several cracks from the diagonal. The lateral of 560.6 kN defined as the peak load was achieved in the loading group of 25.6 mm displacement. The concrete crumbling began to occur at the bottom of the boundary of the specimen. As the further lateral displacement progressed, the existing longitudinal seam between the
cast-in-place part and the PC panel occurred the vertical crack as well as the concrete crushing began to take place from the bottom of the wall as shown in Figure 6. The lateral force was decreased to below 85% of the peak load of the specimen. The test was terminated and the failure photograph of Specimen W-2 was illustrated in Figure 7.

![Figure 6. Local concrete crushing for Specimen W-2.](image1)

![Figure 7. Crack and damage distributing for Specimen W-2.](image2)

3.1.3. Specimen W-3

There were no obvious cracks observed occurring on the Specimen W-3, when the horizontal force was applied. As an increase in the lateral displacement, the lateral force grew up linearly. During this stage, no apparent deformation could be observed. As the horizontal load arrived at 200 kN, a continuous crack took place from the bottom of the wall. The horizontal force arriving at yield shear force 432.2 kN was defined as the yield strength and the corresponding lateral displacement of 7.5 mm was considered as the yield displacement. As an increase in the cycles, the existing deformation became aggravated and more locations suffered from the crack. Afterward, the maximum load began to develop as can be observed from the change in slop of the load-displacement curve. As the further lateral displacement progressed, the some concrete crushing could be observed from the bottom of the specimen as well as no obvious cracks proceeded to taking place as illustrated in Figure 8. The lateral force was decreased to below 85% of the maximum load of the specimen and the test was stopped as the specimen had arrived at the ultimate bearing capacity and avoided suffering a complete failure. The failure photograph of Specimen 3 was shown in Figure 9.

![Figure 8. Local concrete crushing for Specimen W-3.](image3)

![Figure 9. Crack and damage distributing for Specimen W-3.](image4)

3.2. Damage and Failure Mode

From Section 3.1, detailed specification can get access to observation. All specimens had experienced similar failure patterns, which included the process of initial flexural cracks developing into flexure-shear cracks, local concrete crushing at the two boundaries of the specimen, and crack propagation of the PC panel. The damage process can be considered as three major phrases: the elastic phrase, the yielding developing phrase, and the failure phrase. That detailed process achieved in every phrase was summarized as follow.

The elastic stage started from the initial application of lateral load until the yields occur in the specimen. In this stage, the specimen performed almost in an elastic manner. No cracks occurring or any other obvious physical distortion could be noticed.
The yielding developing step denoted by the scope started from the yielding to arriving at the ultimate load of the specimen. The first horizontal crack was seen at the bottom of the specimen. The crack proceeded to developing obviously and propagated to more areas with the horizontal load increasing.

The failure stage initiated from the ultimate force to reaching the failure pattern of the specimens. As the horizontal load increased, the existing crack was greatly developed and aggravated. New cracks could be found at more areas. There was little crushing concrete seen at the corner of the wall and the major fracture continued to develop. Finally, the test came to an end on account of the specimen losing its capacity to resist the lateral load and the specimens were subjected to a complete failure.

4. Analysis and Discussion

4.1. Force-Displacement Curves

The behavior of the shear wall is attached critically importance to maintaining the response of the structure, and is largely relied on the load-displacement response. The hysteresis force-displacement curve is achieved by the lateral force and the horizontal displacement. The hysteresis loops is shown in Figure 10.

Additionally, the ductility capacity can be observed from the hysteresis loops. The shape of the curves is steady and plentiful without a remarkable pinching effect. The conclusion achieved from the hysteresis loops is drawn below:

1. Plumper loops can be observed in Specimen W-2, Specimen W-3 than in Specimen W-1. This difference can be explained that the boundary of the wall is reinforced by the confined boundary elements to suffer from a lager moment-induced compressive stress, which delays the occurrence of fracture and failure. Thus, the confined boundary elements are conductive to more stable and excellent seismic behavior, which is logical and reasonable.

2. The constructional detail of the PC panels has an active effect on the shape of the hysteresis loops. The Specimen W-3 with the notch of panels leads to more stable behavior than that of Specimen W-2. This can be explained that the notch of panels like joint motor strengthens the bound and reduces the weakness existing in the connection between confined boundary elements and the wall, which in turn, enhances the integrity and deformation capacity of the wall as well as improves the ductility capacity of the structure. Therefore, a more steady performance can be obtained.

When the force arrives at the maximum load during the following cycle, the peak point of the following loop is hooked up to the point of the maximum load of the previous loop. The same process come to an end until the failure point is arrived at. In this procedure, an skeleton curve was defined and instituted for each of the positive and negative loading direction as illustrated in Figure 11. The test value including initial stiffness, yield strength, and peak load, which was recorded in Table 3 could be obtained from the skeleton curve.

For all specimen, it can be found that variation tendency of skeleton curve is identical. Additionally, it can be observed that the specimens with confined boundary elements exhibit excellent structural performance. However, the Specimen W-3 achieved the richest ultimate bearing capacity. This is because the bond strength between the prefabricated panels and cast-in-place concrete was reinforced by setting the gap like the joint motor. This is logical and expected as the entire wall with joint motor was subjected to a larger moment-induced axial force when suffering from the lateral load.

### Table 3. Summary of test results.

| Specimen | $F_y$ (kN) | $\Delta_y$ (mm) | $F_m$ (kN) | $\Delta_m$ (mm) | $F_u$ (kN) | $\Delta_u$ (mm) | $\mu$ | $F_m/F_u$ |
|----------|------------|----------------|------------|----------------|------------|----------------|-------|-----------|
| W-1      | 372.4      | 6.4            | 445.3      | 19.9          | 378.5      | 24.0           | 3.9   | 1.18      |
| W-2      | 456.2      | 7.3            | 560.6      | 25.0          | 463.4      | 35.0           | 4.8   | 1.21      |
| W-3      | 432.2      | 7.5            | 556.4      | 30.0          | 444.1      | 40.0           | 5.3   | 1.25      |

Note: $F_y$ is the yield load; $\Delta_y$ is the yield displacement; $F_m$ is the displacement at peak load; $F_u$ is the ultimate load; $\Delta_u$ is the ultimate displacement; $\mu$ is the ductility factor.
4.2. Stiffness Degradation

During the cyclic test, it can be found that the stiffness of all specimens decreased, which is caused by the cumulative damage. The larger deformation under cyclic loads gets access to being exhibited and other structural members can be resulted in extensive damage, when the specimens are with a larger degree of stiffness deterioration.

Stiffness $K_i$ is defined as the ratio of the averaged maximum strength equivalent to the sum of the absolute values of the positive peak lateral load and the negative peak lateral load to the corresponding averaged lateral displacement at the displacement of $X$ including the displacement under the positive load and the negative load, as illustrated in Eq.(1) respectively. The stiffness $K_i$ can be considered as the averaged secant stiffness of the specimens, which is located at different displacement levels. Stiffness degradation ($B_{i0}$) is calculated from secant stiffness at displacement of $X$ ($K_i$) to the initial stiffness ($K_0$) as illustrated in Eq.(2).

$$K_i = \frac{|F_i| + |F_{-i}|}{|X_i| + |X_{-i}|}$$  \hspace{1cm} (1)

$$B_{i0} = \frac{K_i}{K_0}$$  \hspace{1cm} (2)

Where $F_i$ represents the peak lateral load, which is under the $i$ th cycle at the displacement of $X$; uniformly, $X_i$ is also considered as the corresponding displacement of $i$ th cycle. Stiffness at different displacement levels is obtained from Table 4.

The $K_i$-$X$ curves of all specimens are illustrated in Figure 12. The stiffness deterioration curves are made comparison under the cyclic lateral load, and the some conclusions can be drawn as follows:

Almost similar stiffness degradation performance can be observed for all specimens. However,
some slight difference could be found among the specimens. Comparison made between Specimen W-2, and Specimen W-3, Specimen W-3 possessed more steady and gradual stiffness degradation.

The cyclic secant stiffness of the specimens exhibited stable and gradual degradation behavior from the yield point to the failure point as shown in Figure 12. It should be mentioned that the obvious stiffness degradation was observed after the specimen start to yield and the steep drop in stiffness can be accounted for by the cumulative plastic deformation. Generally, the shape of curves was stable and plentiful without a notice stiffness mutation observed.

Confined boundary elements have a great influence on the stiffness. Compared with Specimen W-2 and Specimen W-1, the stiffness degradation of the later is stepper than that of the former owing to the fact that confined boundary elements alleviates the energy absorption capacity and leads to the gradual decrease in the stiffness.

![Figure 12. Rigidity degradation curves against drift ratio.](image)

### Table 4. Summary of stiffness attenuation coefficient results.

| Specimen | $K_0$ (kN·mm$^{-1}$) | $K_c$ (kN·mm$^{-1}$) | $K_y$ (kN·mm$^{-1}$) | $K_m$ (kN·mm$^{-1}$) | $K_u$ (kN·mm$^{-1}$) | $B_{c0}$ | $B_{y0}$ | $B_{m0}$ |
|----------|----------------------|----------------------|----------------------|----------------------|----------------------|----------|----------|----------|
| W-3      | 127.63               | 83.43                | 68.47                | 17.88                | 17.88                | 0.654    | 0.536    | 0.140    |
| W-5      | 136.67               | 95.80                | 59.78                | 22.44                | 15.39                | 0.701    | 0.437    | 0.164    |
| W-6B     | 138.98               | 111.74               | 52.32                | 19.19                | 13.60                | 0.804    | 0.376    | 0.138    |

Note: $K_0$ is the initial elastic stiffness; $K_c$ is the secant stiffness when first crack was observed; $K_y$ is the yield stiffness; $K_m$ is the stiffness corresponding to maximum lateral load; $K_u$ is the stiffness at ultimate load; $B_{c0}$ is the ratios of secant stiffness when first crack was observed ($K_c$) to the initial stiffness ($K_0$); $B_{y0}$ is the ratio of yield stiffness ($K_y$) to the initial stiffness ($K_0$); $B_{m0}$ is the ratio of the secant stiffness corresponding to peak load ($K_m$) to the initial stiffness ($K_0$).

4.3. Ductility

Ductility represents the post-yield deformation ability of the structure, which is considered as the ability of the structure to experience the large plastic deformation from the yielding point up to the failure point without apparent loss of strength. Ductility ratio is a key element to evaluate the behavior of ductility under the cyclic lateral load. The capacity of the specimen to develop plastic deformation without extensive damage is used to be represented by ductility ratio. The yield displacement $\Delta_y$, yield load $P_y$, ultimate displacement $\Delta_u$, ultimate load $P_u$ and ductility ratio $\mu$ are listed in Table 3. The following observation can be made:

It could be observed that ductility of all specimens was displayed from the Table 3. The ratio of the ultimate displacement to the yield one ranges from 3.9 to 5.3. The content deformation capacity of specimen is exhibited, which stands for developing plastic deformation without extensive damage after the yielding of the specimens.

The ratio of the ultimate displacement to the yield displacement for Specimen W-2 and Specimen W-3 are higher than Specimen W-1. Therefore, the confined boundary elements has a significant effect on the deformation capacity of wall and conduces to a increase in the ductility ratio.

4.4. Energy Dissipation Capacity

The energy dissipation capacity is considered as an index of seismic behavior of the structure to be achieved from the force-displacement hysteresis curve, thereby the structure with high energy dissipation capacity exhibiting good seismic performance. As illustrated in Figure 13, the enclosed
area of the load-displacement loop stands for the energy absorbed by the deformation of specimens. The equivalent damping coefficient he is calculated by Eq.(3). The results are listed in Table 5.

$$h_e = \frac{S_{ABC+CDA}}{2\pi S_{OBE+ODF}}$$

Where $S_{ABC+CDA}$ = area enclosed by the hysteresis curve and $S_{OBE+ODF}$ = summation of the triangle areas OBE and ODF.

As might be expected, the energy dissipation of the specimen, which could be calculated by the area $S_{ABC+CDA}$, gradually grows up with an increase in displacement amplitudes of the hysteresis curves up to the failure point. Comparison between Specimen W-2, W-3 and Specimen W-1, Specimen W-2 and Specimen W-3 shows greater energy dissipation capacity, which demonstrates that the confined boundary elements are able to enhance the wall seismic behavior.

It is should be noticed that the arrangement of notch of PC panels has a minimal effect on the energy dissipation capacity, when comparison is made between Specimen W-2 and Specimen W-3.

![Figure 13. Equivalent damping coefficient calculation graph.](image)

### Table 5. Equivalent damping coefficient.

| Specimen | Displacement | $S_{ABC+CDA}$ | $S_{OBE+ODF}$ | $h_e$ |
|----------|--------------|---------------|---------------|-------|
| W-2      | 1$\Delta_y$  | 1382          | 3081          | 0.071 |
|          | 1.5$\Delta_y$| 2271          | 5038          | 0.072 |
|          | 2$\Delta_y$  | 3674          | 6925          | 0.084 |
|          | 2.5$\Delta_y$| 5289          | 8616          | 0.098 |
|          | 3$\Delta_y$  | 6854          | 10150         | 0.107 |
|          | 4$\Delta_y$  | 9460          | 12642         | 0.109 |
|          | 5$\Delta_y$  | 11320         | 15339         | 0.097 |
| W-3      | 1$\Delta_y$  | 918           | 4630          | 0.032 |
|          | 1.5$\Delta_y$| 2688          | 7832          | 0.055 |
|          | 2$\Delta_y$  | 4280          | 11228         | 0.061 |
|          | 2.5$\Delta_y$| 6511          | 14309         | 0.072 |
|          | 3$\Delta_y$  | 9158          | 17171         | 0.085 |
|          | 3.5$\Delta_y$| 11315         | 19934         | 0.090 |
|          | 4$\Delta_y$  | 13063         | 21769         | 0.096 |

### 5. Summary and Conclusions

To investigate the seismic behavior of the specimens, a series of cyclic tests on three full-scaled specimens were carried out under cyclic loading. Three specimens included precast composite wall using the confined boundary elements, precast composite wall without confined boundary elements, precast composite wall with cutout shape like joint motor of panels. The results of the present study are summarized as follows:
(1) The specimens undergo identical damage patterns and failed in the sequence local cracking of the PC panels and concrete crushing at the boundary of the wall. The force-displacement response illustrated steady and plentiful hysteresis performance.

(2) The structure measures of PC panels had a significant impact on the structural behavior of the walls. Specimen W-3 with joint motor showed more stable hysteresis performance, more gradual degradation of the stiffness and higher bearing capacity than Specimen W-2.

(3) The presence of confined boundary elements could strengthen the restraint to the shear wall and reduce the stiffness degradation of wall. Simultaneously, the confined boundary elements contributed to enhancing the seismic behavior and delayed the occurrence of failure.

(4) Further studies are carried out on the effect of axial load, various methods of vertical reinforcement splicing and the effect of shear span ratio. In addition, optimal design should be taken into consideration until the pattern could be paid well to engineering practice.

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