SEISMIC BEHAVIOR OF PRECAST RC SHEAR WALL WITH STEEL CONNECTORS

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Abstract

Premature damage of the post-cast concrete joints of precast reinforced concrete (RC) shear walls was observed in the past earthquakes due to the poor connection details and stress concentration of the wall-beam-slab joints. To avoid the potential failure, a novel connection method of precast elements featuring steel connectors and high-strength bolts is proposed based on the damage relocation design concept. The boundary beam and squat wall pier are cast together to enhance the integrity of the joints. The steel connectors are embedded in the top and bottom ends of precast shear wall panels and boundary beam-wall piers, thus the precast elements are connected away from the slab elevations. Five 1/3 scaled specimens including four precast RC shear walls with respect to various embedded configurations of steel connectors, and one monolithic RC shear wall were designed and tested under quasi-static cyclic loading. Seismic behavior in terms of damage modes, shear strength and energy dissipation capacity were investigated. The results show that the initial cracks of the concrete are changed from joints to the bottom of the precast shear wall panel, demonstrating the damage relocation effect. Four precast RC shear wall specimens have similar drift ratio versus shear force relationships with that of specimen MSW, showing that the proposed connection method could achieve a monolithic behaviour. The equivalent viscous damping ratio of four precast shear wall specimens is higher than that of specimen MSW up to a drift ratio of 1/100, indicating that the precast shear wall specimens provide superior energy dissipation capacity with the embedded steel members. Among the four types of precast RC shear wall specimens, specimen TPSW-N that the non-full length type is adopted in the embedded steel connectors cannot ensure a stable energy dissipation capacity. Specimen HPSW with the lateral steel flanges at the two sides of shear walls could provide the protective effect for the concrete, and its configuration details show the best seismic performance by comprehensively considering the damage mode, drift ratio versus shear force characteristics and energy dissipation capacity. The proposed connection method moves the critical region outside the week connection zones and increases the redundancy of precast shear walls, which could be an alternative for the conventional post-cast concrete connection scheme.

Keywords: Precast RC shear wall; joints; steel connectors; damage relocation

1. Introduction

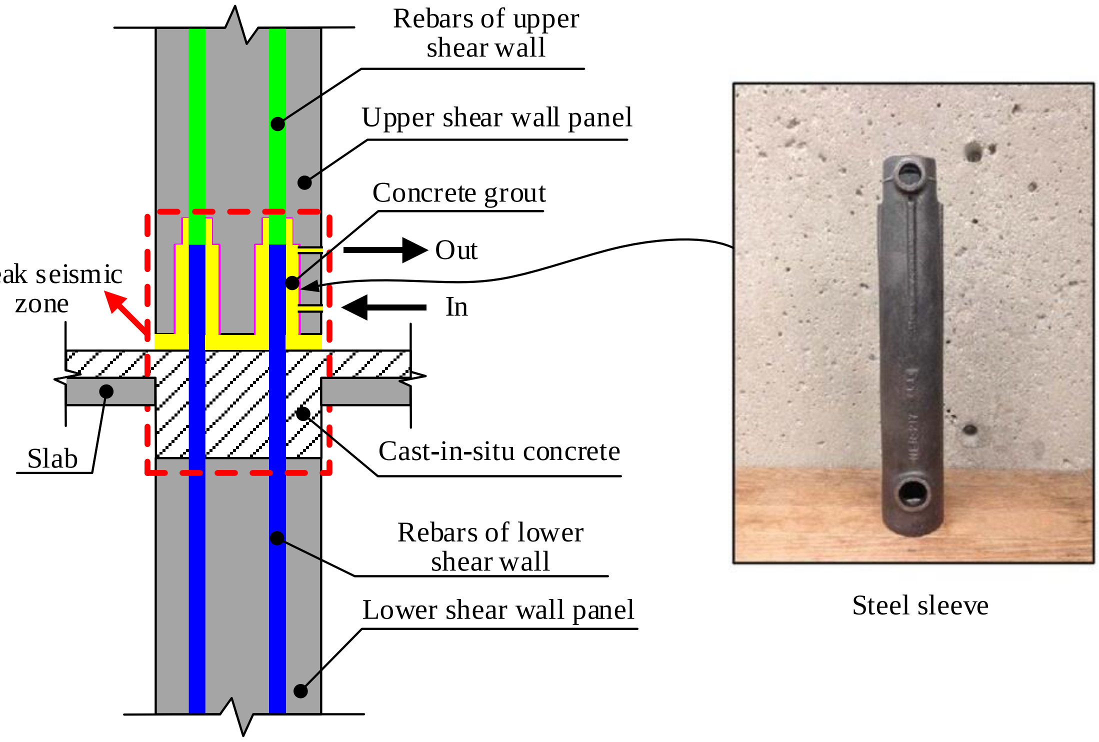
Conventional construction method with cast-in-situ concrete is facing the labor shortage and waste of resources, while adopting a precast structure could be a solution on this issue [1]. The precast structure indicating a new construction process that the structural elements are manufactured in a factory, and erected to a complete structure in the site [2]. Among the precast elements, RC shear walls are widely employed in buildings with its superior seismic efficiency. The connection joints enable to transfer forces among precast elements, which are the most important part of a precast structure.

To date, a large number of researches on the connection joints of precast RC shear walls have been carried out. The reported studies mainly concentrate on the various connection method of steel reinforcing bars, such as grouted sleeve connection, grouted dowel connection, or composite grouted splice sleeve connection [3-7]. Peng et al. [8] studied the cyclic behavior of precast shear walls with a mortar-sleeve connection for longitudinal steel reinforcing bars. Li et al. [9] investigated the seismic performance of the T-shaped precast RC shear wall with grouting sleeves, showing it had similar performances with the cast-in-site specimen. Wu et al. [10] proposed a flexural capacity calculation approach for precast grouted shear wall influenced by joint interface displacements. Aragon et al. [11] proposed a novel short grouted connector of precast concrete structures, which simplified the construction of precast concrete structures. However, it is inconvenient to connect large amounts of reinforcing bars with current methods and hardly ensure the reliability of connections at the complicated construction site. In addition, various grouting defects can be occasionally found in sleeve connections, such as insufficient curing of grout, position deviation of reinforcement and insufficient grouting.

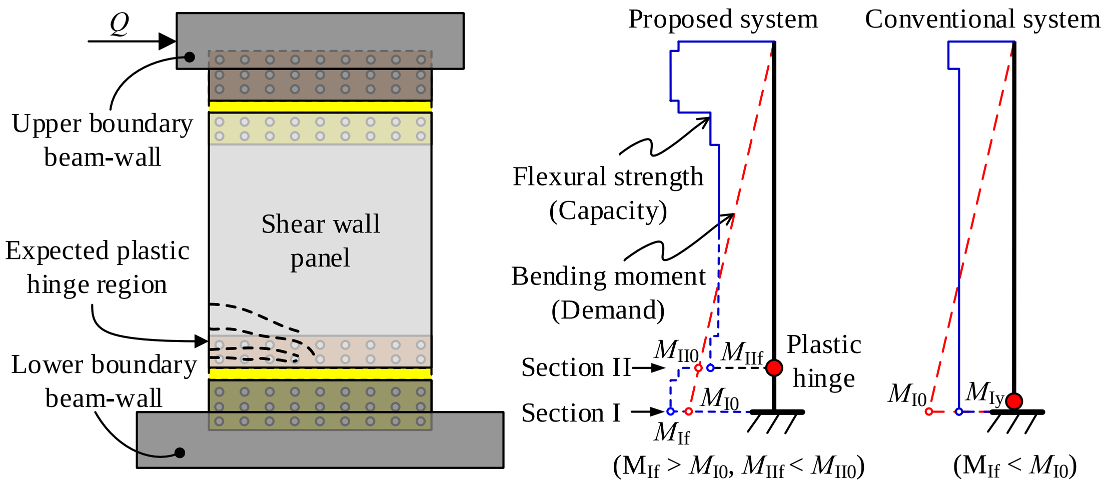
To avoid the potential failure, a new connection method of precast elements featuring steel connectors and high-strength bolts is proposed based on the damage relocation design concept. This paper aims to present an experimental investigation on the seismic behaviour of precast RC shear wall with the proposed connection method. Five specimens, including one monolithic RC shear wall and four precast RC shear walls with respect to various embedded configurations of steel connectors, were constructed and tested under quasi-static cyclic loading. Seismic behavior in terms of failure mode, shear strength, damage relocation, and energy dissipation capacity was investigated.

2. The new precast RC shear wall system

Conventional precast RC shear walls are connected at the slab elevations of structure with post-cast concrete. As illustrated in Fig. 1(a), rebars of upper and lower shear wall panels are connected with non-shrink and high-strength concrete grout. Practically, different grouting defects may be resulted in grouted sleeve connections, owing to some uncertain reasons such as bubbles, clogging and leaking during construction procedures. In addition, insufficient curing of grout and position deviation of reinforcement can also bring the grouting defects [12]. Meanwhile, the joints subject to the maximum internal forces. These factors induce the connection joints to be the weak seismic zones, resulting in the premature failure of joints. A new connection method for the precast RC shear wall system based on damage relocation design concept was proposed. As plotted in Fig 1(b), the precast elements, including upper/lower boundary beam-walls and shear wall panel, are connected by using high-strength bolts through the embedded steel connectors. The steel connectors are embedded in the top and bottom ends of precast shear wall panels and boundary beam-wall piers, thus the precast elements are connected away from the slab elevations. It is noted that two pairs of steel connectors are employed near the ends of the shear wall, which corresponds to the antisymmetric moments of the shear wall. The boundary beam and squat wall pier are cast together ensuring the integrity of the joints. It is expected that the plastic hinges could be transferred to the bottom/top of the shear wall panel. Additionally, the internal forces at the connection section are lower than that at the slab elevations, meaning a higher safety margin. The proposed connection method enables the precast elements to be assembled conveniently because the connection details are made up of steel, which has been widely employed in the steel structures with its high strength and superior workability.



(a) Conventional grouted sleeve connection



(b) Damage relocation design concept

Fig. 1 – The proposed precast RC shear wall system

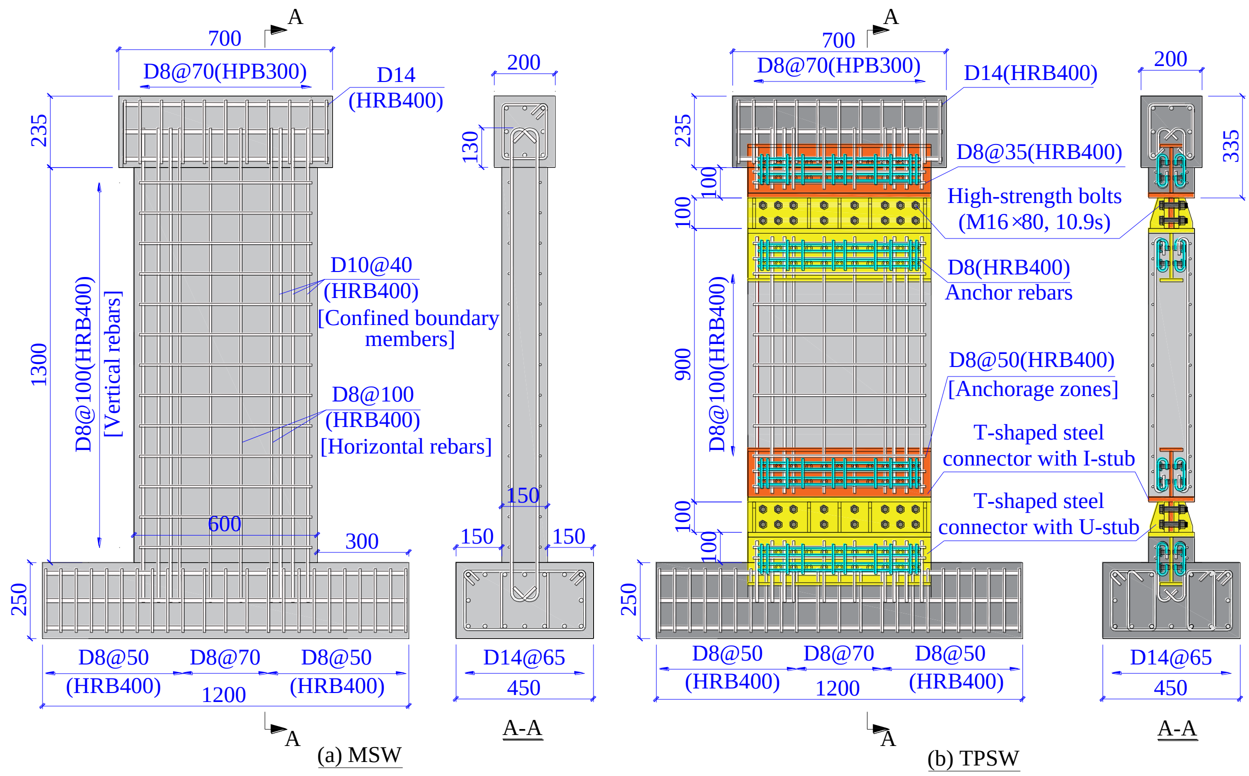
3. Experimental program

3.1 Test specimens

Five 1/3 scaled specimens including one monolithic shear wall and four precast RC shear walls considering different configurations of the embedded part of steel connectors were designed and constructed. Namely, specimens TPSW, IPSW, HPSW, TPSW-N, which featured that the T-shaped, I-shaped, H-shaped, and T-shaped with non-full length type embedded plates welded headed stud groups were used for the steel connectors, respectively. For the purpose of comparison, the monolithic shear wall, named MSW, with identical dimensions was constructed as the reference. Specimen TPSW was used to investigate the anchoring effects with flange and headed stud groups, while specimen IPSW was adopted to explore the anchorage capacity with only headed stud groups. Specimen HPSW was employed to study the protective effect with lateral flanges. Specimen TPSW-N was tested to study the necessity of steel connectors with full-length type. Fig. 2 shows the dimensions and details of specimens. All the specimens were constructed with a concrete grade of C30. D8 (a diameter of 8 mm) rebars with a spacing of 100 mm, which corresponds to a reinforcement ratio of 0.67% in horizontal and vertical directions were adopted for the shear wall panels [13]. D10 rebars with a spacing of 40 mm were designed as the reinforcements at confined boundary members of shear walls. D8 rebars with a spacing of 50 mm were used in anchorage zones to ensure a sufficient embedment capacity of headed stud groups. Specimen MSW was designed with a width of 600 mm and an effective height of 1300 mm. Among the precast shear wall specimens, the steel connectors were made up of a thickness of 10 mm of the embedded steel plate and welded headed stud groups with a spacing of 50 mm. An embedment depth of 150 mm was adopted for the steel connectors. The precast elements were connected by using an I-stub of steel connector insert into the U-stub of another steel connector and fastened with high-strength bolts. Steel stiffeners with a thickness of 10 mm were applied to the steel connectors with U-stub to avoid the buckling of steel plate under vertical loadings according to [14]. In addition, the steel connectors were painted once it manufactured to make it corrosion-resistant.

3.2 Test setup and layout of instruments

The test setup is shown in Fig. 3. The bottom beam was fixed on the strong base of the reaction frame with screws, and a 50t jack was applied between the bottom beam and reaction beam to prevent the specimen from moving horizontally. A 100t jack was installed to provide the vertical axial loading. İt was connected on the top beam of reaction frame with roller support, ensuring the top beam could moving freely. A 50t hydraulic actuator was applied to provide the horizontal cyclic loading. Noted that the actuator was defined as positive (+) in push direction, while a negative (-) in pull direction.



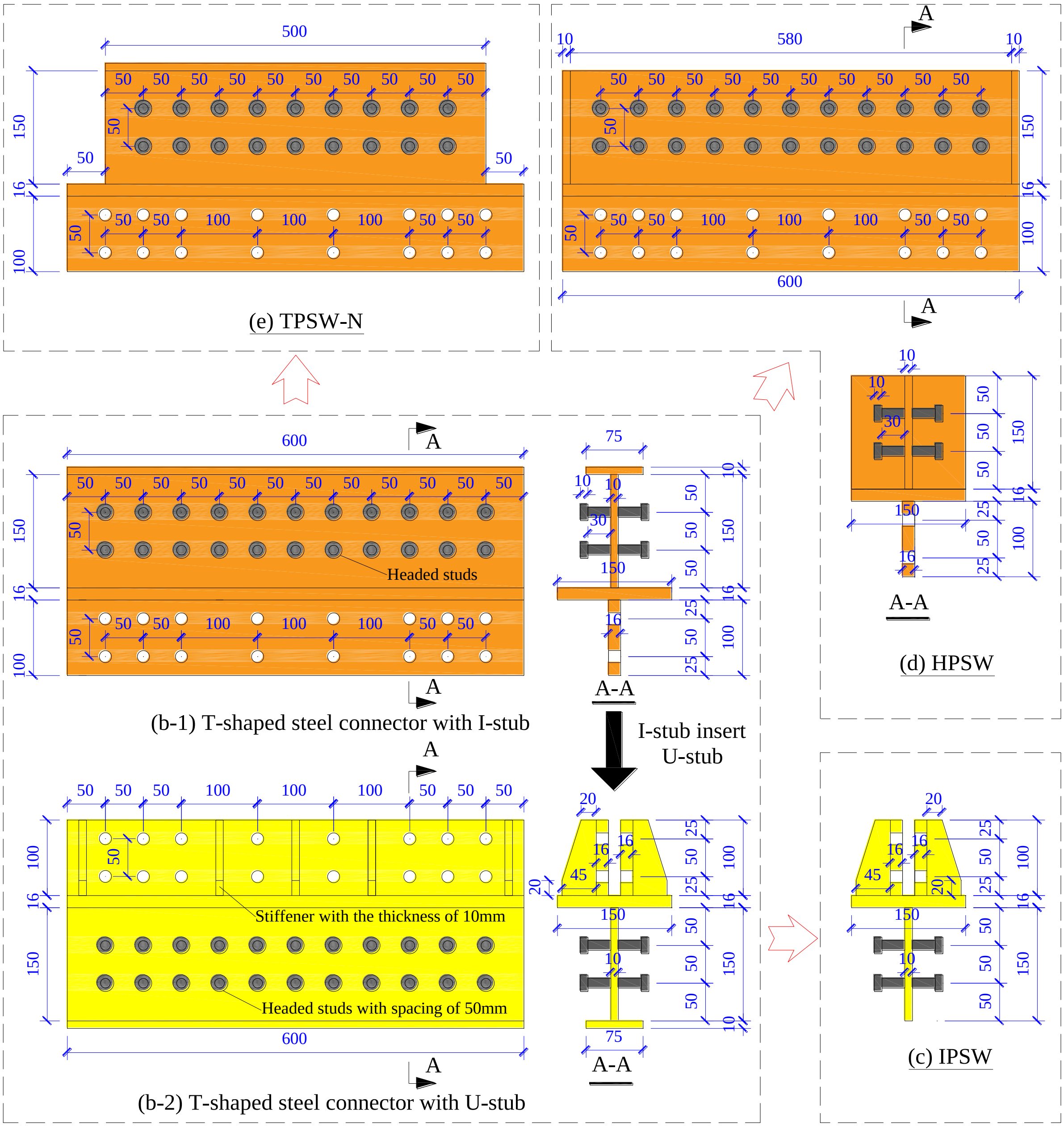


Fig. 2 – Dimensions and details of specimens (units in mm)

Fig. 4(a) shows the layout of displacement sensors. Displacement sensors with pinned ends were used to monitor the deformation components of the shear wall. The shearing deformation could be monitored with the displacement sensors in diagonal directions, while the bending deformation was measured with displacement sensors in vertical directions. Displacement sensors with spring-type (d1-d4) were applied to measure the lateral displacement along with the shear wall height. Two displacement sensors (d5and d6) were used to monitor the sliding and moving of the bottom beam. Fig. 4(b) shows the layout of strain gauges. The strain gauges were installed on the rebar to measure the strain distribution of rebars. Two strain gauges (left and right) were set on one gauge point, and the strain results were obtained from the average values of these strain gauges. It should be noted that the strain gauges were set in identical monitoring positions between the MSW specimen and precast shear wall specimens to justify the damage relocation design concept.

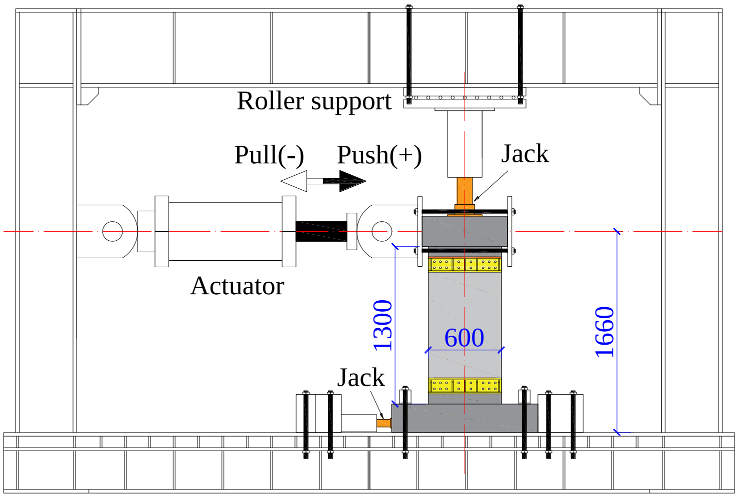
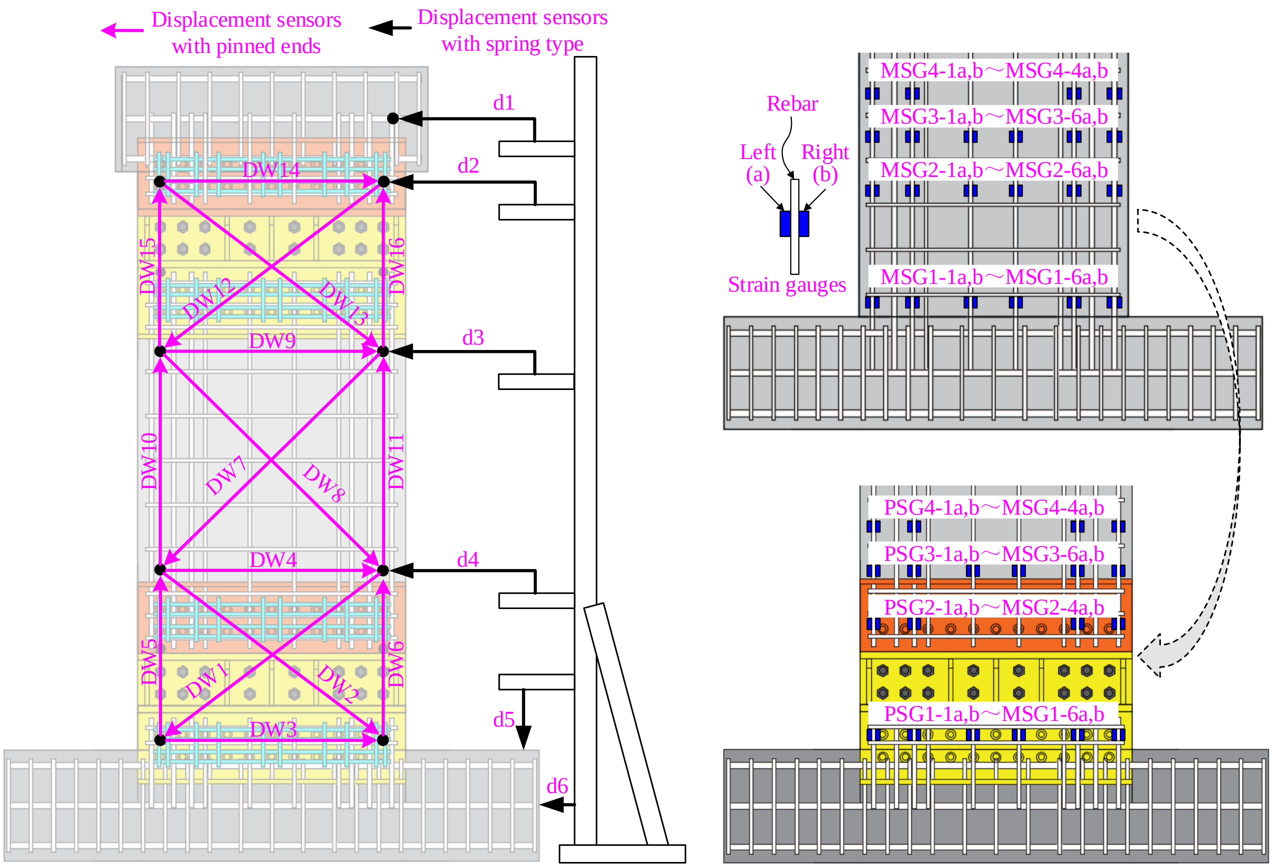


Fig. 3 – Test setup



(a) Displacement sensors (b) Strain gauges

Fig. 4 – Layout of instruments

3.3 Loading protocol

Fig. 5 shows the loading protocol. A force-displacement combined controlled method was applied to the specimens. The vertical loading, corresponding to an axial compression ratio of 0.05, was applied to the specimen with the vertical jack firstly. Then the horizontal loads were performed with a force-controlled method to check the loading devices. The cyclic loadings were further carried out with displacement controlled method under the constant vertical loads. The loading displacements were repeated twice in each loading stage, up to a drift ratio of 1/37. In addition, the loading would be finished as the maximum shear force decreased by 15% of the peak value. It should be noted that a drift ratio of 1/100 corresponds to the limit value of the shear wall under rare earthquake action [15].

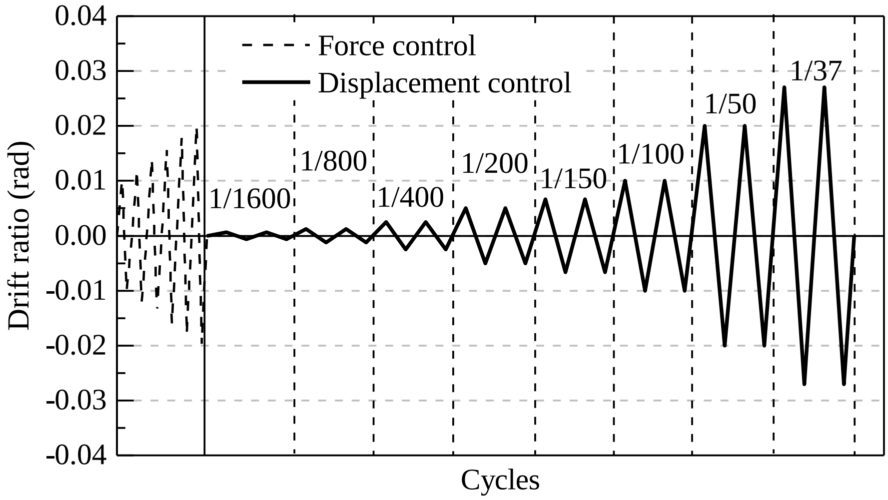


Fig. 5 – Loading protocol

4. Test results

4.1 Damage of specimens

As the loading was carried out, the cracks of concrete were identified and sketched at the peak loading displacements and unloading positions of the first loop in every loading stage. The crack width of concrete was determined by a standard line width ruler. When it was loaded to a drift ratio of 1/100, a large number of concrete cracks could be observed on the specimen MSW and it was mainly distributed on the base of the shear wall panel. However, as for the precast shear wall specimens, slight concrete cracks were developed and it arose at the bottom of the precast shear wall panel initially. It can be found that the concrete damage was transferred from the shear wall-beam joints to the bottom of the precast shear wall panel.

Fig. 6 shows the damage of specimens as the loading was finished. The blue lines represent that the cracks are formed as it is loaded in a positive direction while the red lines mean they are developed in a negative direction. Concrete cracks were mainly presented on the bottom of the shear wall panel, indicating a plastic hinge was formed, and it showed a flexural-dominated failure mode. Compared with specimen MSW, the damages of precast shear wall specimens were significantly decreased. Among the precast shear wall specimens, the concrete cracks were mainly distributed at the precast shear wall panel. It can be also observed that the cracks tend to develop from the zones of embedded steel members, which provides the anchorage effect for the steel connectors. Minor cracks were developed on the lower squat beam-wall pier, indicating that it possessed high strength with the embedded steel connectors and dense steel reinforcements. In addition, noted that the concrete crack of the lower beam-wall pier of specimen HPSW did not occur, inferring that the lateral flange of steel connectors could provide the protective effect for the concrete. It can be found that the precast shear wall specimens were damaged gradually, and the huge brittle failures did not occur during the whole loading process.

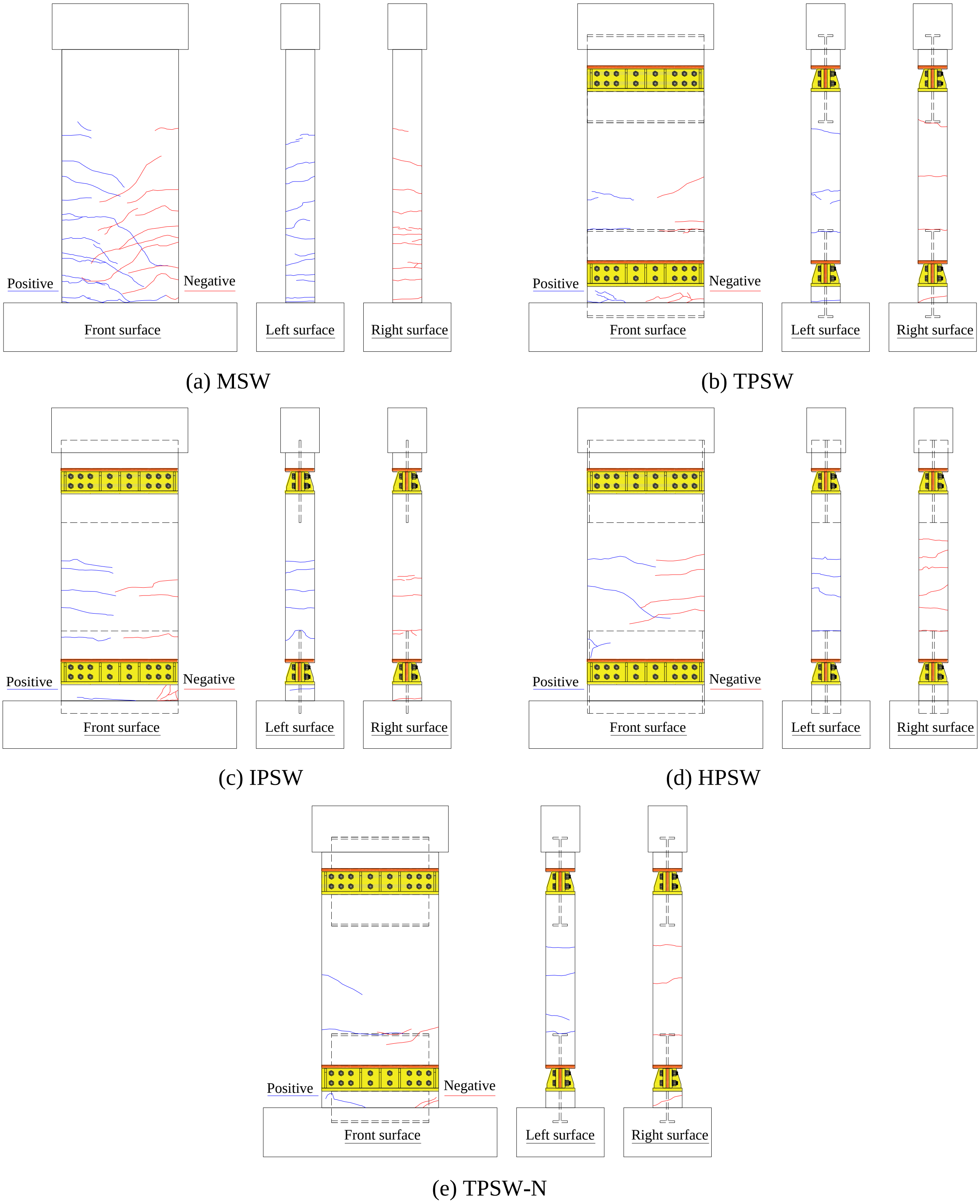
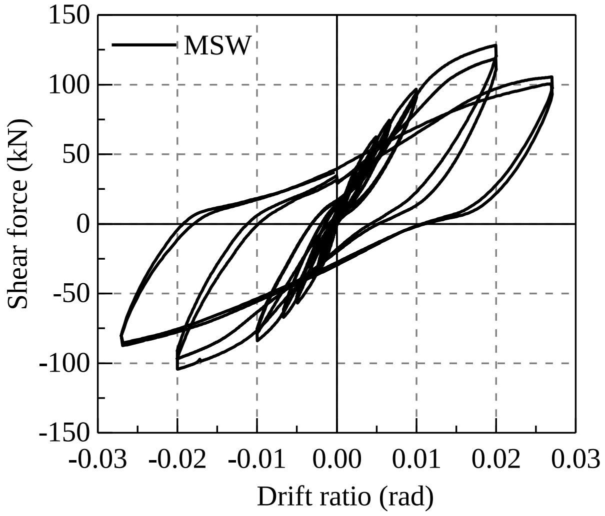


Fig. 6 – Damage of specimens

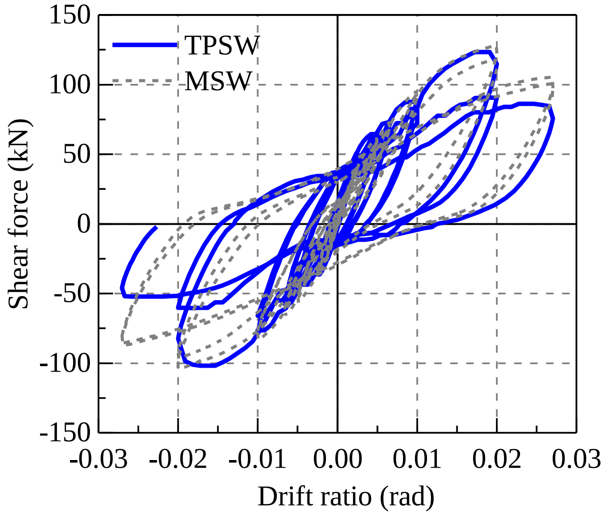
4.2 Drift ratio versus shear force relationship

Figs. 7(a)-(e) show the drift ratio-shear force hysteretic curves of five specimens and Fig. 7(f) shows their skeleton curves. It can be observed that four precast shear wall specimens possess similar drift ratio versus shear force relationships with that of specimen MSW, meaning that the connection methods of precast RC shear wall system can provide efficient stiffness and strength. The maximum shear force of specimens MSW, TPSW, IPSW, HPSW and TPSW-N is 128.24 kN, 123.23 kN, 134.20 kN, 114.46 kN, and 118.38 kN, respectively. The maximum shear force of precast shear wall specimens specimen HPSW is decreased by 10.7% compared to that of specimen MSW. It should be noted that specimen TPSW was only loaded up to the first loop of the last stage because the shear force was sharply decreased compared to the previous loading stage. It can be observed that specimen HPSW has a more stable bearing capacity among the precast shear wall specimens. The shear force of specimen HPSW is 111.47 kN as it is loaded to a drift ratio of 1/37, meaning that the shear force of specimen HPSW is only decreased by 8.4%.

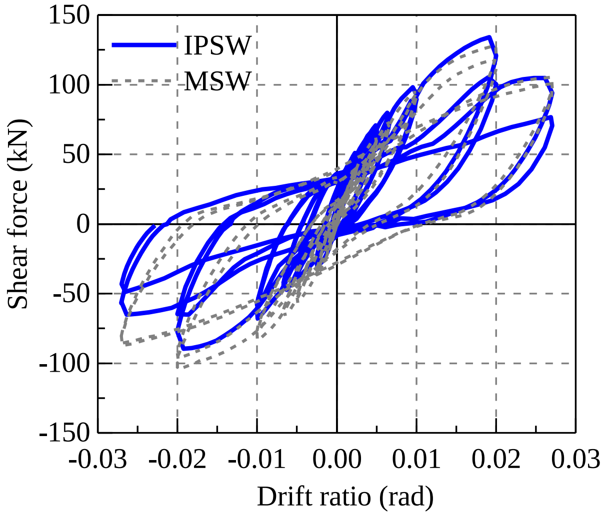
The initial secant stiffness can be obtained by using the calculation that the shear force divided by displacement at the fist loading stage. The initial secant stiffness of specimens MSW, TPSW, IPSW, HPSW, and TPSW-N is 14.31 kN/mm, 19.83 kN/mm, 23.41 kN/mm, 22.18 kN/mm, and 18.95 kN/mm, respectively. It can be found that the initial secant stiffness of precast shear wall specimens is increased with the contribution of embedded steel connectors and reinforcements.



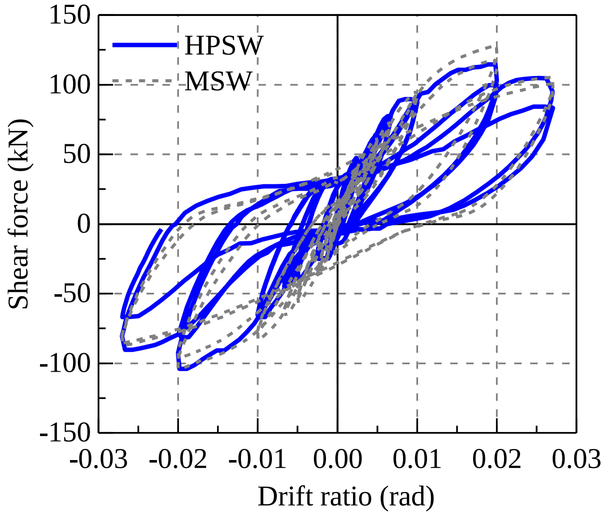
(a) MSW



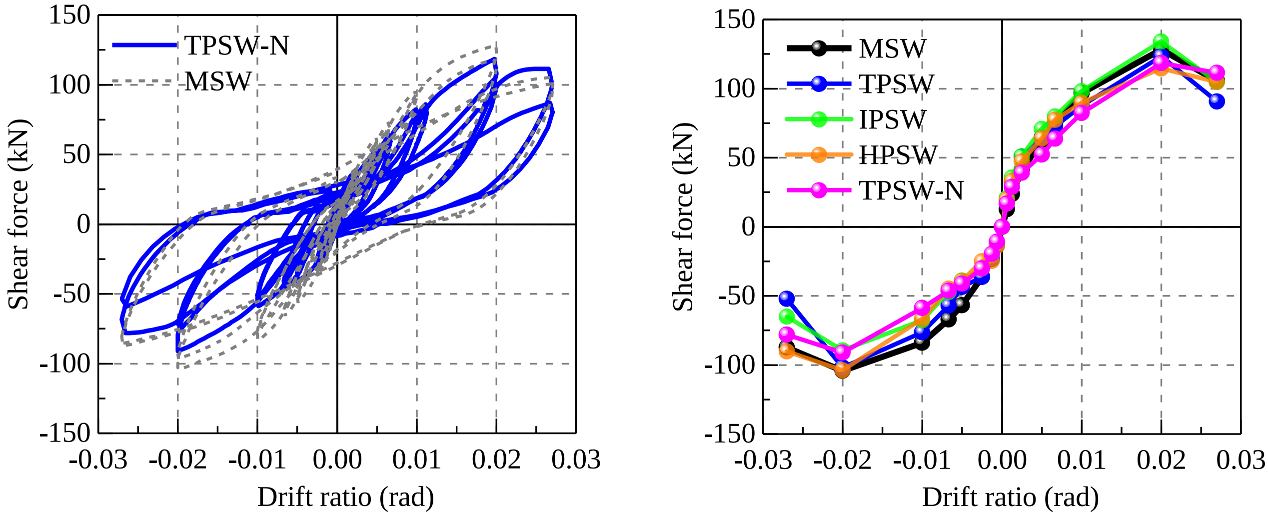
(b) TPSW



(c) IPSW



(d) HPSW



(e) TPSW-N (f) Skeleton curves

Fig. 7 – Drift ratio versus shear force relationship

4.3 Energy dissipation capacity

The energy dissipation capacity is an important index to measure the seismic performance of the structure [16]. The equivalent viscous damping ratio can be used to evaluate the energy dissipation capacity of members. The equivalent viscous damping ratio h_{eq} [17] can be obtained from Eq. (1).

$$h_{eq} = \frac{E_D}{4\pi E_S} \tag{1}$$

where E_D is the energy dissipation at every first loop of loading stage, E_s is the elastic strain energy at the corresponding loading displacement.

Fig. 8 shows the equivalent viscous damping ratio at every loading drift ratio and Table 1 lists its values. It can be found that the equivalent viscous damping ratio of four precast shear wall specimens is higher than that of specimen MSW up to a drift ratio of 1/100, indicating the precast shear wall specimens have superior energy dissipation capacity. The equivalent viscous damping ratio of specimens MSW, TPSW, IPSW, HPSW, and TPSW-N is 6.91%, 14.54%, 11.25%, 11.47%, and 9.54% under a drift ratio of 1/100, showing that specimens TPSW, IPSW and HPSW can provide almost two times of energy dissipation capacity than that of specimen MSW. Among the four precast shear wall specimens, specimen TPSW-N demonstrates a lower equivalent viscous damping ratio after a drift ratio of 1/150, meaning that the embedded steel connectors with non-full length type can not ensure a stable energy dissipation capacity.

Table 1 – The equivalent viscous damping ratio

Drift ratio (rad)		1/1600	1/800	1/400	1/200	1/150	1/100	1/50	1/37
MSW	h_{eq} (%)	3.70	5.43	6.49	7.77	5.78	6.91	13.11	19.08
TPSW	h_{eq} (%)	5.53	6.20	8.08	15.52	13.68	14.54	17.63	15.86
IPSW	h_{eq} (%)	5.58	5.95	10.15	13.31	11.95	11.25	13.93	15.60
HPSW	h_{eq} (%)	7.07	9.05	14.11	16.07	11.22	11.47	15.62	15.71
TPSW-N	h_{eq} (%)	5.35	6.82	11.73	14.61	10.20	9.54	11.52	12.83

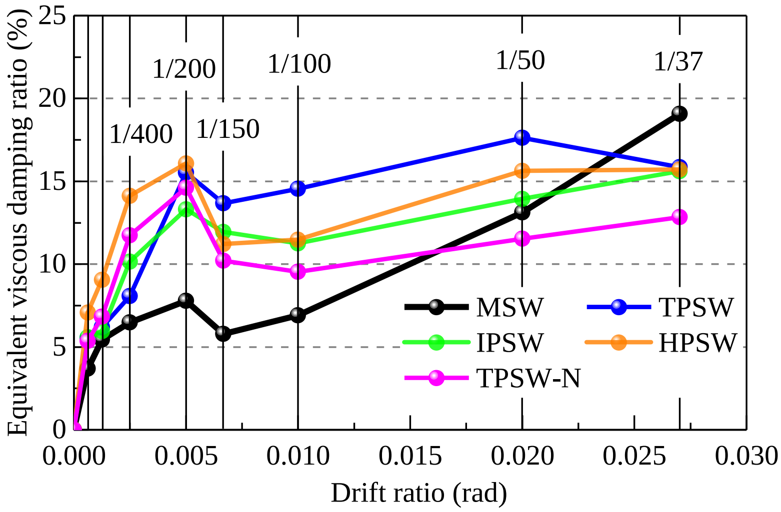


Fig. 8 - Drift ratio versus equivalent viscous damping ratio relationship

5. Conclusion

Five shear wall specimens were constructed and tested under quasi-static cyclic loadings in this work. The observations, damage of specimens, drift ratio versus shear force relationships and energy dissipation capacity were investigated. The following conclusions can be obtained.

(1) Compared with specimen MSW, slight concrete cracks are developed and it arises at the bottom of the precast shear wall panel initially. The concrete damage is transferred from the shear wall-beam joints to the bottom of the precast shear wall panel, demonstrating the damage relocation effect.

(2) Four precast RC shear wall specimens have similar drift ratio versus shear force relationships with that of specimen MSW, showing that the proposed connection method could achieve a monolithic behaviour.

(3) The equivalent viscous damping ratio of four precast shear wall specimens is higher than that of specimen MSW up to a drift ratio of 1/100, in which specimens TPSW, IPSW and HPSW can provide almost two times of energy dissipation capacity than that of specimen MSW. The precast shear wall specimens provide superior energy dissipation capacity with the embedded steel members.

(4) Among the four types of precast RC shear wall specimens, specimen TPSW-N demonstrates a lower equivalent viscous damping ratio after a drift ratio of 1/150, meaning that the embedded steel connectors with non-full length type cannot ensure a stable energy dissipation capacity. The concrete crack of the lower beam-wall pier of specimen HPSW does not occur, indicating that the lateral flange of steel connectors could provide the protective effect for the concrete and its configuration details show best seismic performance by comprehensively considering the damage mode, drift ratio versus shear force characteristics and energy dissipation capacity.

6. Acknowledgements

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