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**Seismic Resistant precast concrete building structures: Precast concrete shear walls**

by

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# CHAPTER 1. INTRODUCTION

Precast concrete buildings expanded across the globe and became a staple of modern construction. Precast concrete construction is an ideal construction method with multiple benefits. The materials of precast concrete are not expensive. The construction used precast concrete elements pre-produced in the plant, which effectively decreased the construction time and reduced the pollution for the in-site environment. The precast concrete construction reduces the construction cost in different ways, such as reducing the number of laborers, reducing the construction period, and using molds can be used up to hundreds of times.

However, before the 1990s, the development of precast concrete construction in the seismic region was limited by two major reasons. One was the potential safety hazard of precast concrete buildings in the seismic region. Several worldwide historical accidents of precast concrete buildings with inadequate seismic-resistant design brought both terrible life and economic losses. Beginning in the early 1960s, some seismic activities happened in 1964 Alaska, 1971 San Fernando, 1977 Rumanian, 1985 Michoacán (Mexico), 1988 Armenian, 1989 Loma Prieta, 1994 Northridge, 1995 Kobe (Japan), 1999 Kocaeli (Turkey), 2001 Bhuj (Indian), 2009 L’Aquila, 2011 city of Van (eastern Turkey). Another reason that limiting the precast concrete buildings in the seismic region was no standard design provisions/codes, and the lack of experimental data of the performance of precast concrete buildings under seismic activities or Simu-seismic tests to support the safety guarantee to them.

The performance of the precast concrete buildings under seismic activities relies on different seismic resisting systems, such as frame systems, floor diaphragm systems and shear wall systems. Within each system, the connections between different components play a significant role in terms of the seismic resistance. This report includes a literature review of each of these three systems and the structural properties of four different shear wall systems. Besides these points, comparisons between four different shear wall systems were included.

# CHAPTER 2. LITERATURE REVIEW OF SEISMIC RESISTING SYSTEMS THAT WERE APPLIED ON PRECAST CONCRETE BUILDING

In 1987, Hawkins and Englekrik [2] illustrated the concepts of the development of the precast concrete building. At that time, Japan was attempting to make precast concrete structures perform as monolithic structures, while the United States recognized the differences between precast concrete buildings and monolithic constructions. The United States paid more attention to readily constructible, dry units, rather than connecting elements with cast-in-place concrete and/or other floating glue. Besides, at that time, the US was interested in shifting the precast concrete construction from panel structures to frame structures. Precast concrete connections require post-yield deformation or energy dissipation, stress demand was high but elastic behavior was anticipated, with low moment demand. Connections using epoxy-based grout joints and post-tensioning were in the development phase in both Japan and New Zealand. After, researchers subjected to different seismic resisting systems did consecutively.

## Frame system

In the 1990s, a research program named PRESSS (Precast Seismic Structural System) was initiated by the United States and Japan, aiming to develop effective seismic structural systems for precast buildings and to prepare seismic design recommendations for incorporation into the "model" building codes at that time [3]. The reasons for the conduct of the PRESSS program can be classified into two classes, as Priestley illustrated in 1991, commercial and safety considerations. For commercial consideration, the precast concrete building at that time as a very competitive construction comparing to others (e.g., steel structures), could benefit economic development due to its characteristics of inexpensive materials, rapid erection, potential innovative ability on design, and construction (e.g., computer-aided manufacture). Besides, the utilization of precast concrete buildings in seismic regions of North America had tremendous commercial prospects. For safety consideration, however, the US did not have any mature and reliable seismic design method nor design code/specification for multi-story precast construction due to a lack of support of real experiences and test data.

The PRESSS program was conducted by several different individual researchers, which were then studied together to make further progress. The PRESSS program focused on the frame systems and panel systems with ductile connection concepts [4]. In 1993, Priestley et al. [5] investigated a concept of connecting precast concrete beam and column elements by using beam prestressing tendons unbonded through the joint and for a distance on either side. The connection could cure itself after a large seismic displacement (e.g., the structure would return to its original position without residual displacement and the initial stiffness would be restored). Ductility demands for the connection were less than fully bonded connection.

In 1996, according to Priestly [6], four generic types of connections had been developed by the PRESSS program: non-linear elastic connection systems; tension-compression yield connection systems; shear yield connection systems; energy dissipating friction connection systems. And the PRESSS program was focusing on the following researches: ductile connections for precast concrete frame systems [4]; behavior of a six-story office building subjected to moderate seismicity [7, 8, 9]; seismic response evaluation of precast structural systems for various seismic zones and site characteristics [7]; dynamic response of precast concrete frames [10]; precast frames connected with unbonded post-tensioning [11]; high-performance fiber reinforced concrete (FRC) energy-absorbing joints for precast concrete frames [12]; seismic behavior and design of double-tee panel precast systems [6]. After studying and analyzing the outcomes from those individual experiments, the final phase of the PRESSS project was an attempt to establish two structural systems designed from the knowledge framework developed in the previous PRESSS projects, a precast frame building, and a precast panel structure. However, due to the limited funding, the precast panel building project was put aside.

In March 1999, after 10 years from the initial of the PRESSS program, a 60 percent scale prototype five-story precast concrete building was designed and erected by previous design concepts to examine their suitability. According to Nakaki et al. [13], Four different seismic frame systems in one direction to resist longitudinal lateral loads and a jointed shear wall system in the orthogonal to resist transverse lateral loads were designed in the building. The frame systems were different because of four different ductile connection systems: Tension-Compression Yielding (TCY) gap connection; TCY connection; Hybrid connection; and pretensioned connection. Wall systems utilized unbonded post-tensioning at the center of each panel. The objective was to make the wall have the recentering ability when the seismic load is removed so there will be no residual drift after a design-level earthquake.

In October 1999, the building was tested under seismic input levels equivalent to at least 50 percent higher than those required for Uniform Building Code Seismic Zone 4. by Priestley et al. [14]. There was no crack in the wall for the shear wall system, but some crack was developed at the base connection to the foundation, which can be easily repaired. Therefore, the overall damage to the shear wall systems was minimal. The energy dissipated by the wall system is considerable. The damage to the building in the response direction of the frames is much less than can be expected of the equivalent reinforced concrete structure, subject to the same drift level. The results showed that the prestressed frame performed well with minor damage.

According to Kurama et al.2018 [22], design code and/or provisions of Frame systems are largely based on the ACI-318, which provide emulative design specifications applicable to special moment frames; ACI-317.1-05 provide the design provision of moment frames with jointed connections.

### Floor Diagram System

In the 2000s, researchers focused on the precast floor diaphragms (Fleischman, et al. [15-Fleischman et al.2013]). Earthquakes [16] and researches [17-Fleischman et al.1998, 18-Wood et al.-2000] found that diaphragm damage is another critical factor that causes precast structural failure. According to Kurama et al. [19], the reasons why floor diaphragms failed were also summited as the following 6 points. (1) diaphragm design forces can significantly underestimate the inertial forces that develop in the floor system during strong earthquakes due to the effect of higher modes during the nonlinear structural response (2) nonductile load paths can develop in diaphragms designed with past design approaches; (3) diaphragms can possess complicated internal force paths, leading to combined tension-shear actions on individual diaphragm connectors; (4) nonlinear demands can concentrate at certain key joints; (5) the diaphragm reinforcement may not possess sufficient nonlinear deformation capacity for these demands; (6) physical experiments and post-earthquake observations have highlighted that axial elongation (from nonlinear rotation) in beams of moment frames can result in significant nonlinear demands in diaphragms, causing potential collapse with existing nonductile detailing of support connections and topping slabs.

In 2003, NSF (National Science Foundation) funded a research project called NEES (Network for Earthquake Engineering Simulation) to developing an industry-endorsed comprehensive seismic design methodology for precast/prestressed concrete floor diaphragms [20]. The project was processed by DSDM (Diaphragm Seismic Design Methodology) Consortium, which is formed by three universities with three different research directions: University of California San Diego (UCSD) with quasi-static and shaking table tests; Lehigh

University (LU) with integrating large-scale experiments; University of Arizona (UA) with finite element (FE) and nonlinear dynamic analyses [21-Fleischman et al.2005].

In addition, deficiency of the floor diaphragm design also brought economic disadvantages to precast concrete buildings. After the foundation of unexpected floor diaphragm action caused the seismic failure, precast diaphragms were not allowed to use in the seismic region. To ensure the safety of the floor diaphragm, ACI 318-14 required that all precast floor units need a cast-in-place topping. In 1997, UBC-97 (UBC1997) further only permitted the use of topped non-composite diaphragm (a thick topping with heavy two-way reinforcement), where precast floor units serve only as gravity load-resisting units. As a result, the economic benefit of precast construction was restricted [15-Fleischman et al.2013, 22-Kurama et al.2018].

According to Kurama et al. [22-Kurama et al.2018], two different floor diaphragms had effects on finding potential ways for using them in seismic regions. One is untopped diaphragms (using mechanical connectors only). Another is topped composite diaphragms (mechanical connectors between precast diaphragm units acting in conjunction with a thin topping with mesh or light reinforcing).

In 2013, based on the outcomes of the NEES project, new knowledge on the seismic performance of precast concrete floor diaphragms was created with several different indexes, e.g., stiffness, strength, deformation capacity, internal force paths, force demand, drift, etc. The research results were adopted into ASCE/SEI 7-16, including an alternative diaphragm design force calculation for general construction, new diaphragm design provisions to accompany the new diaphragm design force, and precast diaphragm connection qualification testing protocols [15-Fleischman et al.2013, 19-Kurama et al.2018]. Nowadays, the design of precast floor diaphragms could follow ACI CODE-550.5-18, which is a stand that describes code requirements for the design of precast concrete diaphragms subject to earthquake motions, where used under the design provisions of ASCE/SEI 7-16 Section 12.10.3 and ACI 318, and subject to shear overstrength provided by the connections and reinforcement at joints specified in ASCE/SEI 7-16 chapter 14.

**Shear Wall System**

Structural wall systems are a commonly used lateral load resisting system in the seismic resisting filed with large lateral stiffness and strength, resulting the limited drift during an earthquake, providing a high degree of protection against non-structural damage. The precast shear wall system can be split into two categories: one utilized emulative connection, while another utilized jointed connection. The emulative connections refer to “strong” connections, where their elastic limit will not exceed the design level in satisfying the building’s ductility demands. The emulative precast wall systems aimed to emulate the cast-in-place walls. Jointed refer to “ductile” connections, where the energy dissipation occurred in the connections, thereby contributing to the building’s overall ductility. The jointed precast wall systems do not emulate the cast-in-place precast wall systems.

According to Kurama et al.2018 [22], precast wall systems can also be clarified as single wall systems and coupled walls. Within each system, each wall can comprise a single panel or multiple panels. In a multifaceted panel wall, vertically stacked panels are connected by horizontal joints. Coupling walls consist of two or more vertical walls connected in the same plane by connectors or coupling beams. Coupled beam design can make most of the hysteretic energy dissipation occur in the beam, thus limiting the damage borne by the vertical load-resistant wall element [25].

At the early stage, the platform-type wall system is the main structure of the emulative shear wall system. The horizontal hollow-core slab panel is placed and connected as floor and roof between the vertical panels by strong horizontal connections [23, 24]. As figure 1 shows.

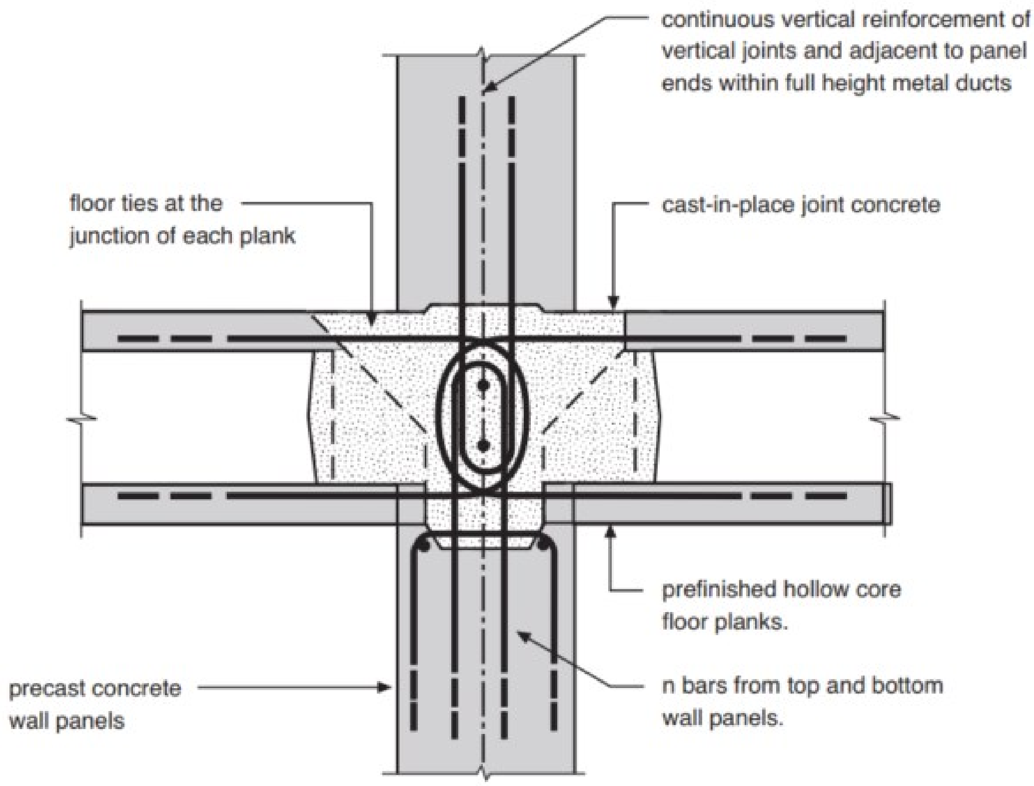



Figure 1. Nearly monolithic precast wall construction horizontal joint of the SCT system [25]

From 1995, the emulative shear wall systems have a form that the precast panels stacking on each other straightforward without floors in between [25]. Connections between the panels included grouted corrugated metal ducts or proprietary grout sleeves to splice deformed mild steel reinforcing bars.  Figure 2 shows the typical grouted wall panel-to-panel connections.

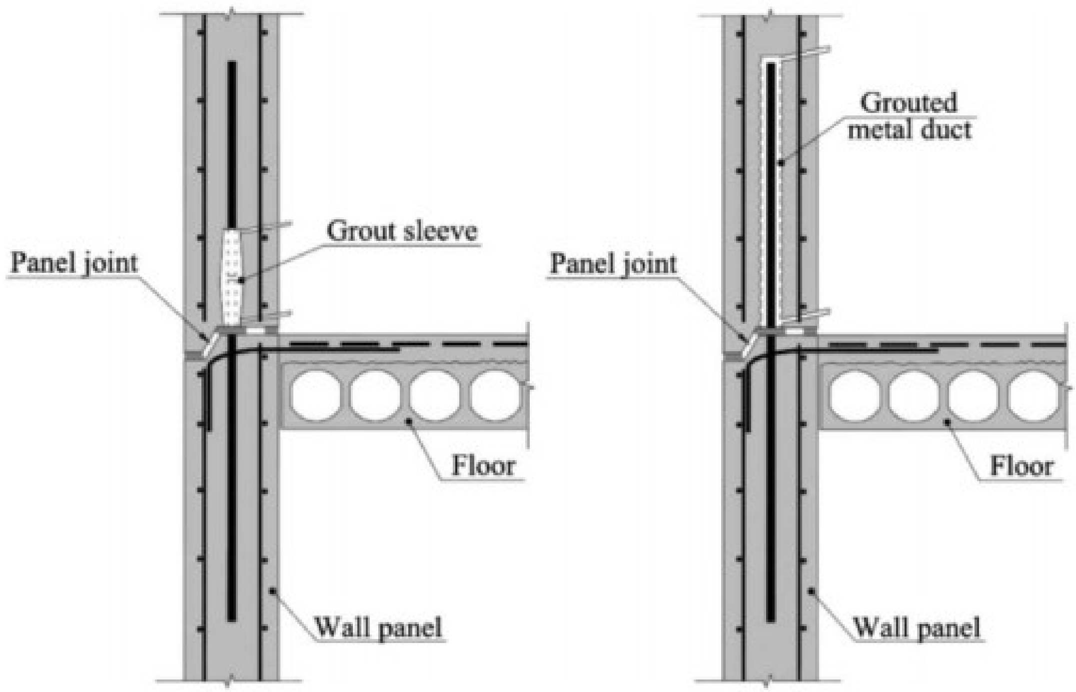



Figure 2. Typical grouted wall panel-to-panel connections [25]

Unbonded Post-tensioned precast concrete wall system is a commonly used jointed wall system with an ability to remain elastic during a design-level earthquake and the re-centering ability after the load is removed, resulting in no residual drift under an expected earthquake. Unbonded post-tensioned precast concrete walls are constructed from post-tensioned prestressed precast wall panels through horizontal joints, using post-tensioned prestressed steel, and are not bonded to concrete. Lateral load resistance is provided by high-strength post-tensioned bars or multiple tendons located in un-grouted pipes. Figure 3 shows the unbonded post-tension wall [26]. Horizontal joints between wall panels can open and close under cyclic seismic loading.

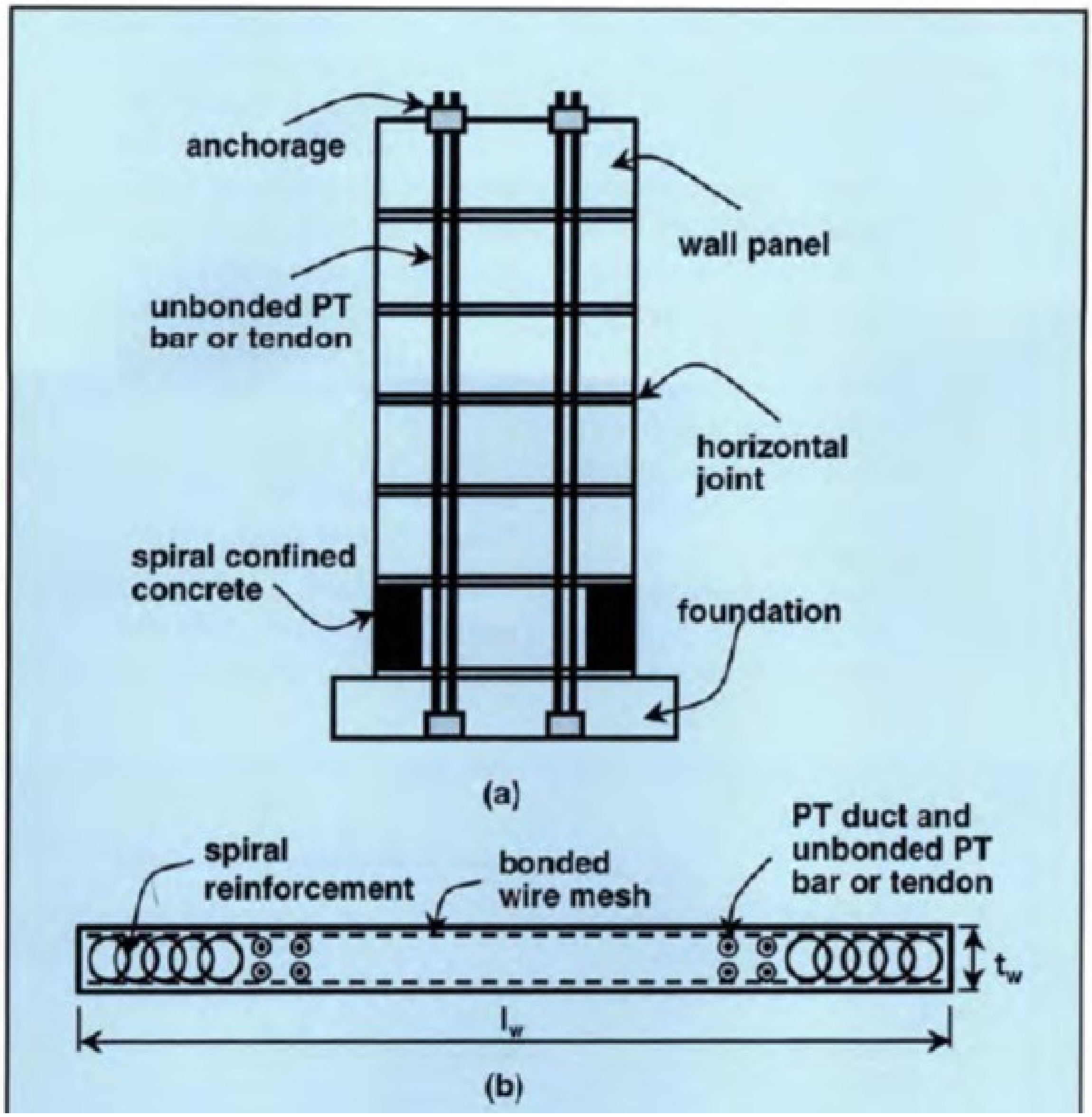



Figure 3. Unbonded post-tensioned wall [26]

Kurama et al. [26]in1999 illustrated the behavior of the unbonded post-tensioned precast walls in detail. The unbonded post-tension wall systems have two behaviors under gravity load and seismic load: Gap opening and shear slip. Both of them happened along the horizontal joints between the vertically stacked panels. The former behavior can be restored by the action of the axial force under the action of the post-tension tendon and the gravity load. The latter behavior is not an ideal lateral displacement mode. The proposed design method can be used to prevent shear slip under severe survivable horizontal ground motion. To avoid the shearing slip along horizontal joints, baseboard-foundation joints' minimum shear slip capacity is required to be greater than the maximum base shear requirement.

The seismic resistant systems should have a sizeable hysteretic energy dissipation capacity to ensure an adequate inelastic structural response. However, the unbonded Post-Tension (PT) connections have a low energy-dissipation ability. To provide PT connections with sufficient energy-dissipation capacity, adding an external energy dissipator is a common way.

Restrepo and Rahman2007[27] added the longitudinal mild steel reinforcement crossing the joint between the walls and the foundation as energy dissipators, adding significant energy dissipation capacity to the unbonded post-tension shear wall system preserving the self-centering response. The experimental adopted quasi-static reversed cyclic loading. After comparison, the wall system without an energy dissipator performed a nonlinear elastic response with a very little energy dissipation; The wall system with energy dissipator performed a typical “flag-shape” hysteretic response, and the energy dissipators have 14% equivalent viscous damping ratios. As figure 4 and 5 shows.

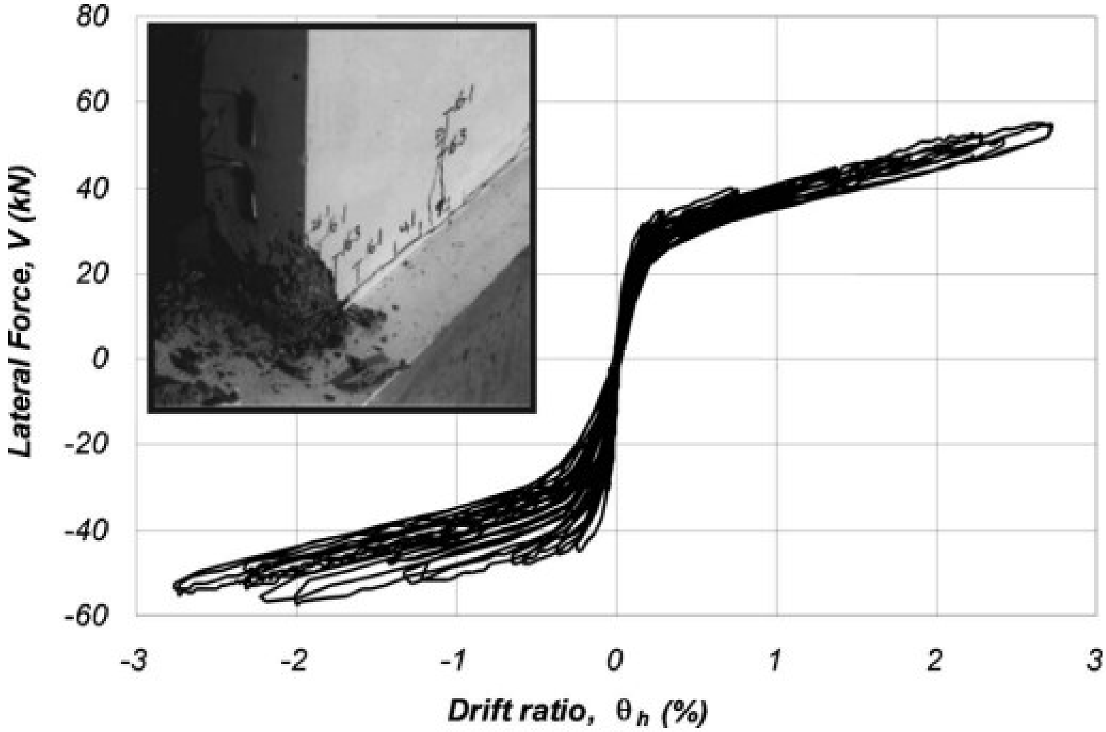



Figure 4. Lateral force-drift ratio response of unbonded post-tensioned shear wall without external energy dissipator [27].

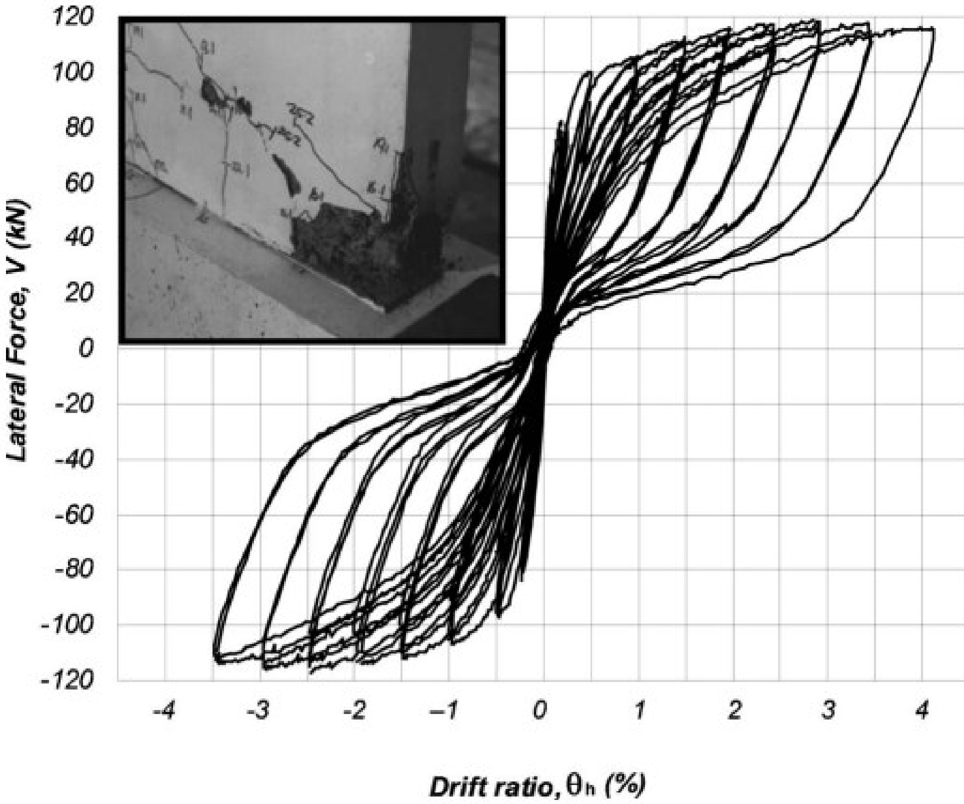



Figure 5. Lateral force-drift ratio response of unbonded post-tensioned shear wall with energy dissipator. [27]

Current shear wall design includes four types:

Ordinary precast concrete structural (shear) walls

Ordinary reinforced-concrete shear walls

Intermediate precast definite structural (shear) walls

Special precast concrete structural (shear) walls

The design code of the first two types of shear walls refers to chapters 1 through 18 of the PCI Industry Handbook Committee [28]. The design code of intermediate precast concrete structural (shear) walls refers to chapters 1 through 18 in addition to chapter 21.12 of the PCI Industry Handbook Committee [28], and also ASCE 7-05. The special seismic design requirement of the special precast concrete structural (shear) wall refers to ACI 318-05, 21.13 21.2.2.3, 21.2.3 through 21.2.7, 21.7, and 21.8.

# CHAPTER 3. PRECAST SHEAR WALL SYSTEMS

In conventional monolithic reinforced concrete buildings, the shear walls can significantly reduce the horizontal sway of the structure due to their high lateral strength and stiffness. Shear walls act as flexural members, act as a primary seismic resisting component of the structure along the in-plane direction, and transfer the corresponding lateral forces to the foundation to ensure the structure's safety. The performances of precast concrete shear walls under seismic activities were affected by multiple parameters, such as lateral strength of shear walls, deformation characteristics of shear walls, and energy dissipation capacity.

The shear walls' performance was affected by supporting soil and footings, stiffness of the diaphragm, relative flexural and shear stiffnesses of the shear walls, and connections. In precast concrete building design, it is common practice to neglect the soil and the footings' deformation and assume that the floor and roof diaphragms act as rigid diaphragms [28]. Rigid diaphragms distribute shears to each shear wall in proportion to the shear wall's relative stiffness. The stiffness of the connections between precast wall panels belongs to the whole stiffness of the precast wall systems. The relative flexural and shear stiffnesses of the shear walls and connections are the most critical factors that need to be considered. The connection can change the contribution of connection displacement in the system, leading to different seismic design parameters, such as energy dissipation and failure drift. In low-raised precast concrete buildings, the connection method between the shear wall and the foundation plays a significant role in the performance of the shear wall.

## Performance of the Shear Wall Used Grouted Sleeve Inserts between Wall Base and Foundation

Grouted joints are widely used to connect precast concrete wall panels to the corresponding foundations. The utilization of metal ducts (connection reinforcement) is necessary to reduce the grouted joints' damage because the bending ability of the grouting joint without mechanical connection is lower than that of the wall panels. The reinforcement from one component would insert into the metal ducts located on another component and then grout in materials. The material of grouting inserts was variable, but with more minor temperature volume change, less shrinkage, and less creep [29].

Seifi et al. [29] investigated the force-displacement behavior of connections with the grouted sleeve of two shear walls that used grout sleeve inserts to explore the effects of the connections to the walls. The extended bars from the foundation were placed inside the grouted sleeve inserts positioned inside the wall panel during the wall panel construction. Two full-scale experiments were conducted with one wall panel reinforced with a single layer of vertical reinforcing and the other with a double layer. Then they measured and compared the performance of the two walls by subjecting them to reverse in-plane cycle loads until the failure of either the connection or the wall panel. The overall load-displacement performance of the precast concrete wall panels was calculated as the sum of the wall panel deformation (shear and flexural deformation) and the connection displacement. Both axial and lateral loads were applied to the wall panels.

### Dimensions of Two Different Shear Walls

Table 1. Dimensions of two shear walls [29]

| | Panel 1 | Panel 2 |
|---|---|---|

| | | |
|---|---|---|
| | (Single reinforcement slayer) | (Double reinforcement slayer) |
| Panel Size | 4000mm height,<br>2000mm length,<br>150mm thickness | 4000mm height,<br>2000mm length,<br>200mm thickness |
| Reinforcement | Reinforced horizontally with a single layer of grade 500 HD12 spaced at 250 mm. the vertical reinforcing bars had a 16 mm diameter and a spacing of 300 mm connected to the top of grouted sleeve inserts. | Reinforced with double layer grade 500 HD12 and a spacing of 240 mm, the vertical reinforcing bars had a 16 mm diameter and a spacing of 300 mm connected to the top of grouted sleeve inserts. |
| Connection rebar between wall panel and the foundation | HD16 rebar with a spacing of 300mm | HD16 rebar with a spacing of 300mm |

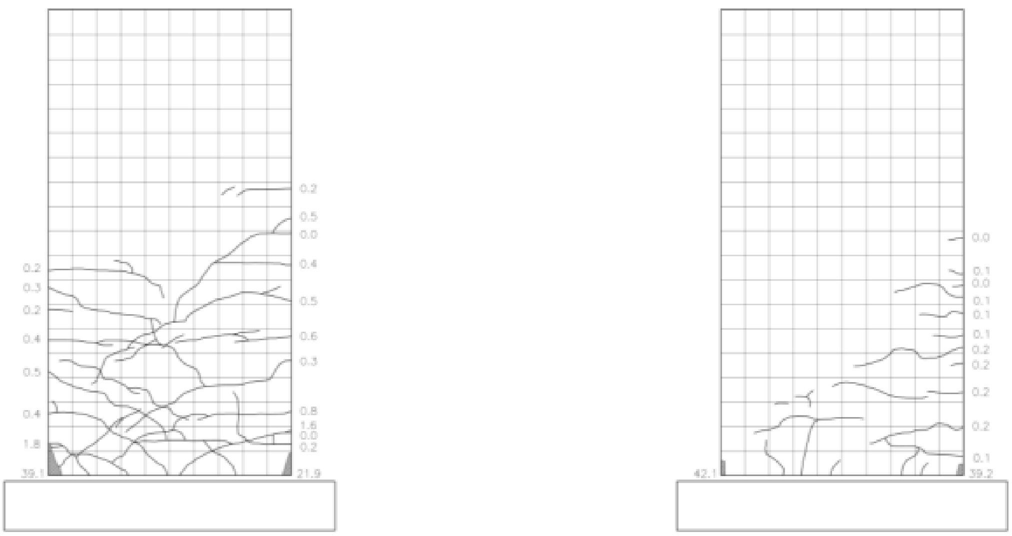
**Comparison Between Two Different Shear Walls**

Two walls were tested under both axial and lateral loads to the wall panels to examine the in-plane seismic behavior of the wall panels connected to a foundation by the grouted sleeve connection. Table 2 and Figure 6 show the comparison of the crack patterns of two shear walls when grout sleeve inserts are located between the wall panel and foundation. This type of connection has two unexpected disadvantages when the inserted reinforcement is subjected to cyclic loads. One is the thread slip effect of connections, and the other is the reinforcement pull-out from the grout. The crack pattern proved that the grouted sleeve shear wall with double reinforcement layer has a better crack pattern than the wall with single reinforcement layer.

Table 2. Comparison between Single reinforcement layer shear wall and double reinforcement layer shear wall in terms of the crack pattern [29]

| Applied drift lever | Single reinforcement layer | Double reinforcement layer |
|---|---|---|
| Blow 0.5% | Two cracks appeared widths of 1.0 mm and 0.3 mm.<br><br>Located at heights of 200 mm and 500 mm from the connection. | No crack happened. |
| 0.5% | New cracks appeared on one vertical edge of the wall panel. On another vertical edge, only increasing the width of existing cracks. | Two cracks with a width of 0.1 mm at the vertical edge of the wall panel at the elevation of 700 mm and 1000 mm from the connection level. And no crack on another vertical edge. |
| 0.75% | New cracks appeared on both vertical edges, a maximum width of 1.4mm. | New cracks appeared on the walls of the maximum width of 0.4 mm.<br><br>And concrete at the compression toe of the wall panel spalled. |
| 1.0% | New cracks appeared on the vertical edge with fewer existing cracks, comparing to another vertical edge. Then, the crack pattern on both vertical edges becomes symmetric. | No new cracks on the wall panel. |
| 1.5% | New cracks appeared with the largest width of 1.8mm.<br><br>Compression toe of the wall panel started to spall. | More extensive concrete spalled.<br><br>Reinforcement pull-out from grouted sleeve inserts occurred which resulted in the closure of many cracks. |

| | | |
|---|---|---|
| 2.0% | Reinforcement pull-out occurred in both extreme grouted sleeve inserts. | No new crack occurred. |
| 3.0 % | Reinforcement pull-out from all other inserts of the connections. | Two connection bars ruptured. The other five connection bars pull out from grouted sleeve inserts. |



(a) Panel 1 (b) Panel 2

Figure 6. Crack patterns of Single reinforcement layer shear wall (panel 1) and double reinforcement layer shear wall (panel 2) [29]

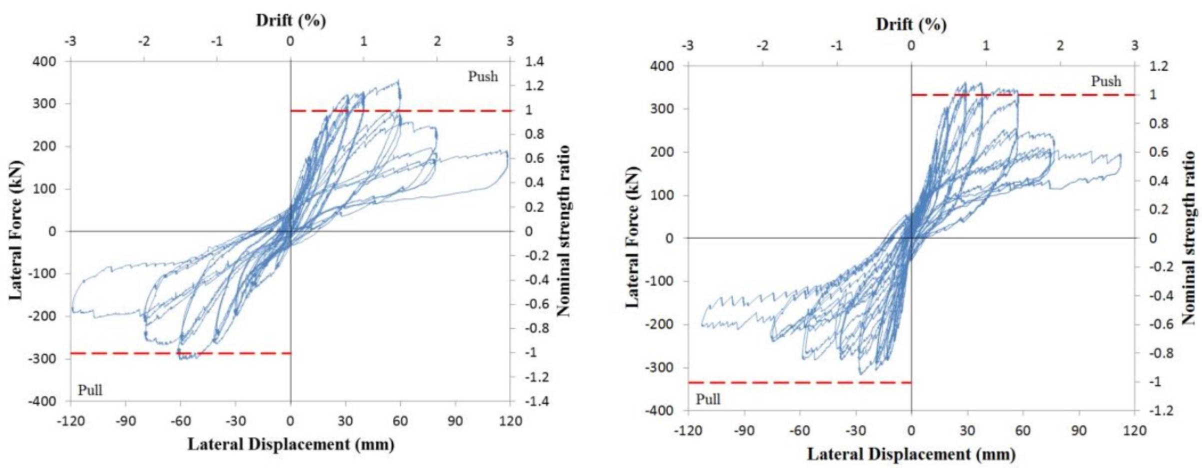
**Analysis of Load-Displacement Curves**

Figure 7 shows the load-displacement curve of both panels. Table 3 shows the comparison of these two panels based on the curves. For the behavior of both panels, maximum lateral forces of different drift levels are different at two sides of the panel. The reason for this behavior was the larger thread slip that occurred on one side of the wall panel than on another side, and it decreased the stiffness of the wall panel on one side of the force-displacement diagram.

Table 3. Comparison between Single reinforcement layer shear wall and double reinforcement layer shear wall of load-displacement response [29]

| Phase | Single reinforcement layer | Double reinforcement layer |
|---|---|---|
| First two cycles | Almost linear elastic behavior | Almost linear behavior |
| third cycle | Large displacement was recorded in one side of the load-displacement diagram of the wall panel. (Thread slip effect) | Almost linear behavior |
| Fourth cycle | inelastic behavior and pinching of the diagram. | inelastic behavior and pinching of the diagram. |
| Drift level of 0.75% | diagram pinching became larger | wall panel reached to the maximum lateral force of the 354 KN and 316 KN in each direction |
| Drift level of 1.0 | diagram pinching became larger | reinforcement pull-out occurred in both extreme connection reinforcement causing a reduction in the lateral force of the wall panel |
| Drift level of 1.5% | The lateral force reached to the maximum magnitude of 350 kN in one side of the load-displacement diagram and 302 kN in the other side of the diagram. | the lateral force decreased to the magnitude of 343 kN and 263 kN in each direction |

| | | |
|---|---|---|
| Drift level of 2.0% | reinforcement pull-out caused larger pinching in the diagram. As loading continued, more reinforcement pulled out from the inserts and it decreased the stiffness of the connection. | two extreme connection bars fractured and caused a significant reduction in the lateral force of the wall panel. The lateral force was 237 KN and 239 KN which were about 67% of the maximum lateral force |
| Drift level of 3.0% | The lateral force was 187 KN which was 52% of the maximum lateral force. | All remaining connection reinforcement pulled out from their inserts in this drift level. |

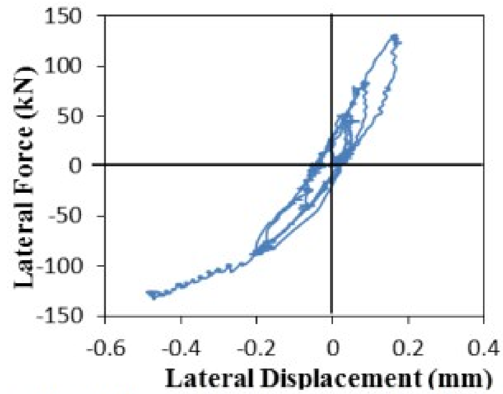



(a) Panel 1 (b) Panel 2

Figure 7. load-displacement curve (hysteresis response) of the wall with single reinforcement layer (panel 1) and the wall with double reinforcement layer (panel 2) [29]

Thread slip effect would cause cracks on the wall early after applying the cyclic lateral load. When the applied drift lever was blowing 0.5%, cracks appeared on the single reinforcement layer wall, whereas no crack happened on the double reinforcement layer wall at this stage. The thread slip effect happened to double reinforcement slayer slab when the drift level reaches 0.5%. The crack at very early stages would cause a considerable stiffness reduction of the wall panel. Figure 8 shows the cracks resulted from the thread slip effect and stiffness reduction caused by thread slip of the single reinforcement layer panel.

(a) Cracks resulted from threaded insert




(b) Stiffness reduction caused by thread slip

Figure 8. Cracks and stiffness reduction caused by thread slip at early stages of loading [29]

The crack width raised on the single reinforced concrete panel is more and wider than the double concrete panel. As the eight stages listed on table 2, the widths of cracks of the single reinforcement layer panel reached 1.0 mm as drift below 0.5%, a maximum of 1.4mm as the drift of 0.75%, a maximum of 1.8mm as the drift of 1.5%. While no crack appears to the double reinforcement layer panel as drift below 0.5%, a maximum of 0.4mm as the drift of 0.75%, no crack as the drift of 1.0%. Cracks decrease the shear stiffness, the fewer and thinner cracks, the better the performance of the shear wall.

## Laminated Slab Concrete Shear Wall

Laminated slab concrete shear walls were constituted by two external prefabricated wall plates, and a concrete filed internal layer called "laminated layer sandwich" between that two external plates, as figure 9 shows.

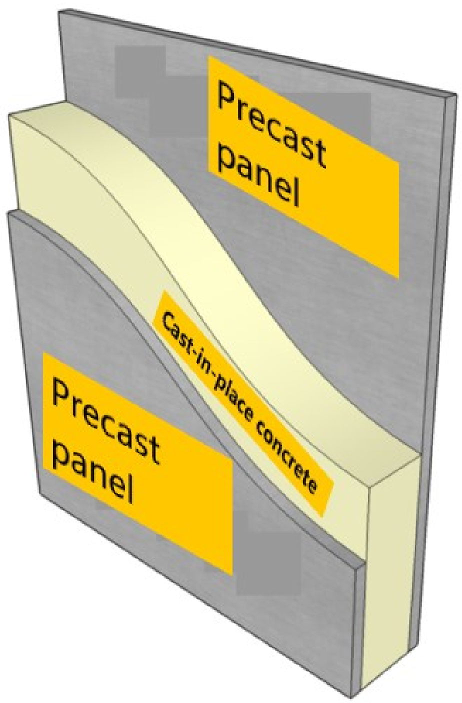



Figure 9. Concept of laminated slab concrete shear wall

A research group [33] tested laminated slab concrete shear walls to verify the similarity between laminated slab concrete shear walls and cast-in-place shear walls. One cast-in-place concrete shear wall, name W1, and two laminated slabs, name W2, and W3, as a dimension of 2800 (height) x 2000 (width) x 200 (thickness) mm was studied. For W2 and W3, two prefabricated wall plates have thicknesses of 50mm each; the laminated layer has a thickness of 100mm. W2 and W3 have two separate walls; each of them has a dimension of 2800 (height) x 1000 (width) x 200 (thickness) mm. Laminated shear wall W2 used a rigid reinforcement connection between two separate panels, whereas W3 used a simple reinforcement connection between two separate panels, as appendix I shows. After applying cyclic load, the degradation tendency of stiffness of laminated shear walls and cast in place shear wall was similar, as figure 10 shows.

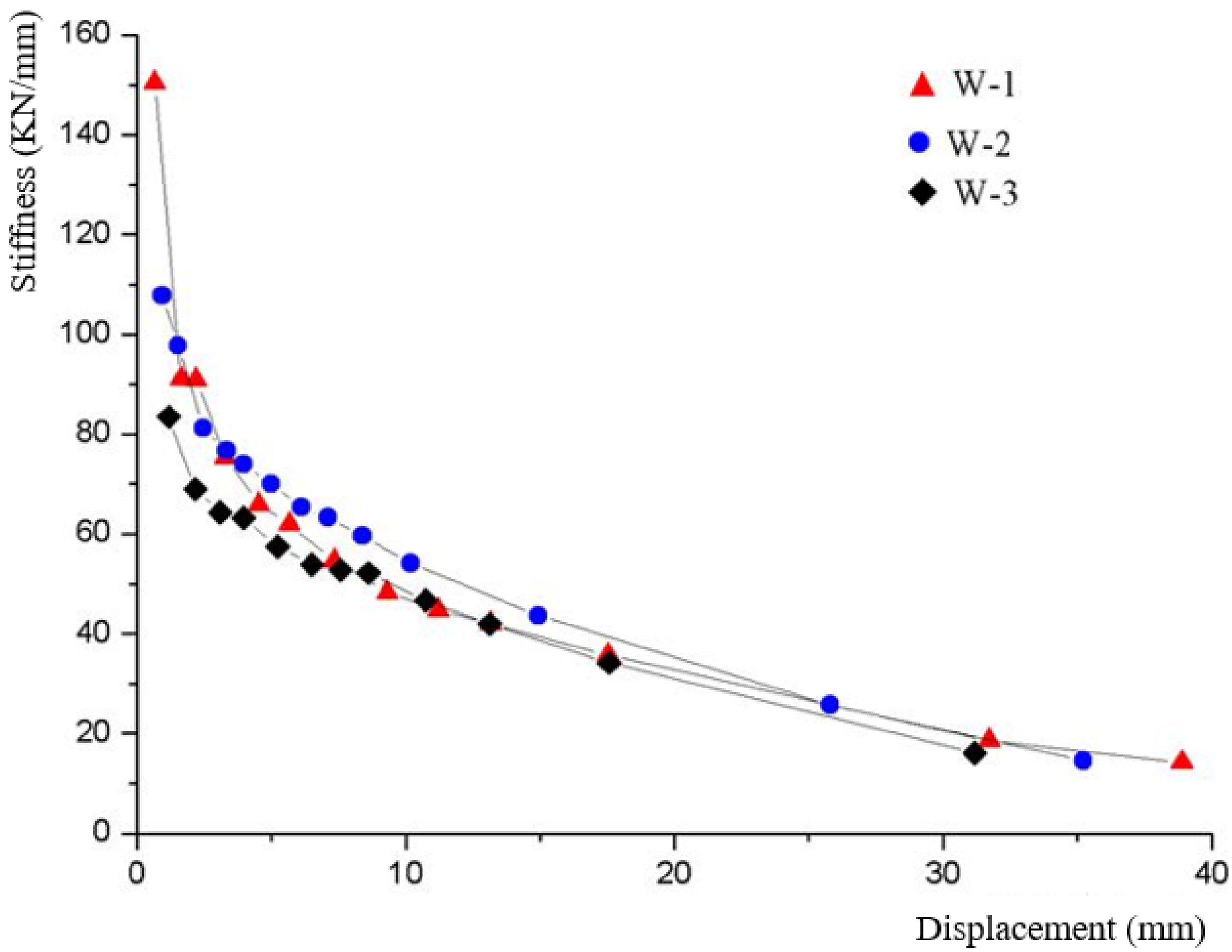



Figure 10. Stiffness degradation curve of W1, W2 and W3.[33]

The damage in W1, W2, and W3 are vertical shear-type damage, in laminated shear wall W2 and W3, overlapping wall plates and the seam is intact. For W1, as the load increases step by step, new cracks continue to appear along with the height of the wall, and new cracks continue to occur in the middle of the early cracks. They continue to develop diagonally below the plate. The width of the crack is about 0.5 mm, both ends of the bottom wall concrete press cracks, peeling, and swelling buckling of longitudinal reinforcement.

Figure 11. Main crack and concrete spall of W1.[33]

For W2, under cyclic lateral load, horizontal cracks appeared from the vertical edge of the wall, develop to the end, all extend obliquely downward, and pass through the middle vertical seam to spread to another prefabricated wall plate extend to the bottom diagonal. The final crack zone height is about 1.4m, which took 50% of the total height of the wall. At the bottom-side part of W2, concrete from the laminated layer spall out with cracks developed. Vertical reinforcement exposed.


Figure 12. Main crack and concrete spall of W2.[33]

For W3, cracks occurred between the bottom wall and the horizontal base construction joints. Vertical cracks happened on both surfaces of the out layer and the new-filled concrete layer. Horizontal cracks are similar to W2. At one side of W3, concrete from one out later and newly filled layer spall out, not much. The separation between the two layers not happened.


Figure 13. Main crack and concrete spall of W3.[33]

**Unbonded Post-Tensioned Precast Concrete Shear Wall**

Unbonded post-tension precast concrete shear walls were known for their self-centering ability in a prefabricated concrete building, thus keeping a condition without serious damage after seismic activity. The precast seismic structural system (PRESSS) research program [Priestley et al., 1999] has investigated that unbonded, post-tensioned shear walls can be used as the primary lateral load-carrying element in regions of high seismicity. A five-story precast concrete building adopted unbonded post-tensioned share walls as its primary lateral load resisting system. The unbonded shear walls presented satisfied with only minor nonstructural damage in the loading direction. Residual drifts in the wall direction after design level excitation did not exceed 0.06% after sustaining a maximum top drift of 1.8% of structure height. PRESSS program recommended that the location of the post-tensioned tendon should near the middle of the wall to protect them from large tensile strains [30].

Erkmen and Schultz [30] investigated the self-recentering mechanisms of precast unbonded post-tensioned shear walls. The group tested an unbonded post-tensioned shear wall with six post-tensioned tendons—test sample as shown in figure 14. Vertical post tendons were placed in the oversized duck without grouting and anchored to the walls at the top and the foundation. The tendons were post-tensioned to 95 ksi, which corresponds to approximately 60% of the average ultimate strength of the tendons. Six post-tensioned tendons were spliced using standard couplers and a layer of high-strength dry-pack mortar with a thickness of 0.75 inches was placed between vertical panels at the horizontal joint for alignment purposes.

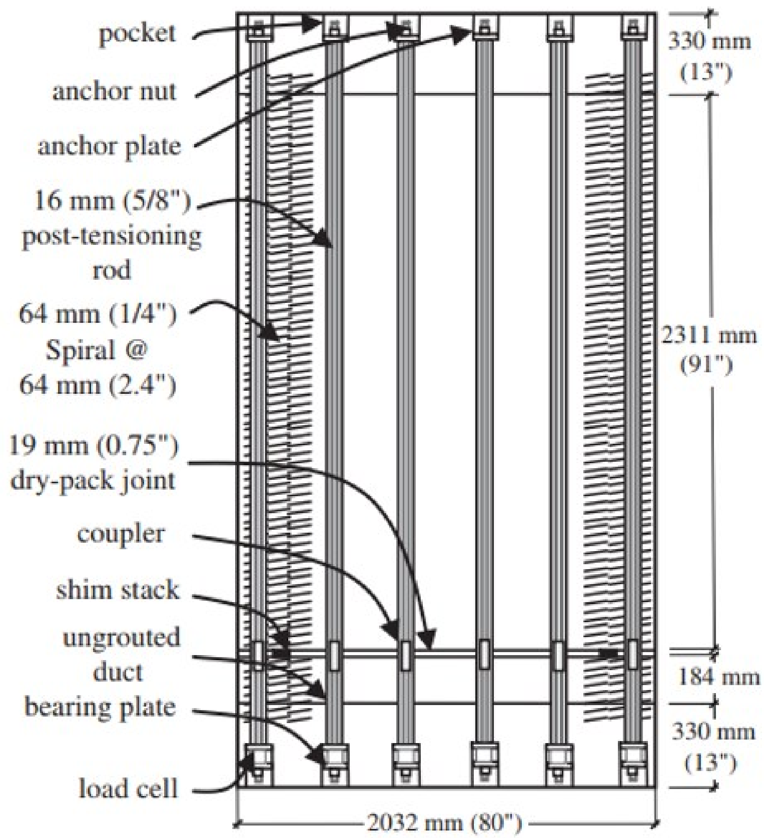



Figure 14. Details of unbonded post-tensioned shear wall spacemen [30]

When the flexural reinforcement is post-tensioned and placed inside oversized ducts without grout (unbonded) can significantly improve the seismic performance of precast concrete shear walls. Because there is no boundary between the post-tensioned reinforcement and the adjacent concrete, the bond stress transferred from the reinforcement will not deliver to the adjoining concrete; thus, the damage on adjacent concrete was avoided [[Cheok et al., 1993; Priestley and Tao, 1993]. Under the lateral force, post-tensioned tendons will transfer the lateral load caused by flexural action across the horizontal joint. The lack of bond between the tendons and the adjacent concrete provides a gap along the horizontal joint. When the lateral load releases, the post-tensioning bars, vertical loads, and concrete compression force lead to a restoring moment that controls and closes the gap.

## Precast Concrete Unbonded Post-Tensioned Shear Wall with End Columns

### Introduction of PreWEC

A new system of precast concrete unbonded post-tension shear wall system (PreWEC) with end columns was introduced by Sritharan et al. [31]. Two end columns with unbonded post-tension tendons inside of them were connected at both sides of the wall panel by O-connectors. This unbonded post-tension concrete shear wall system has a lateral load-carrying capacity similar to a comparable reinforced concrete wall while minimizing damage and providing excellent self-centering capability [31]. Based on Sritharan et al., the columns can undergo relatively small uplift and can be used to transfer gravity load. The end columns in this system can be steel columns, concrete-filled steel tubes, or precast concrete end columns. The wall and columns in the PreWEC system are anchored to the foundation with unbonded-post-tensioning and jointed horizontally using O-connectors along the vertical direction. The O-connectors can improve the energy dissipation ability of the system by undergoing inelastic deformations when undergoing seismic earthquake loading. Figure 15 shows the elevation view of a PreWEC system. Figure 16 below shows the geometric theory of the O-connectors and their deformed shape.

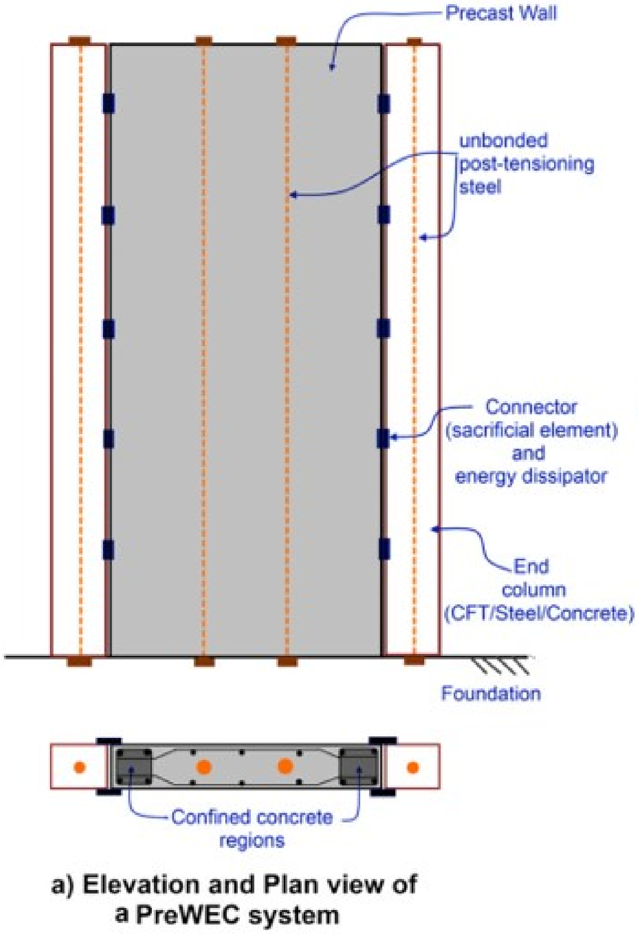



Figure 15. elevation view of a PreWEC system [31]

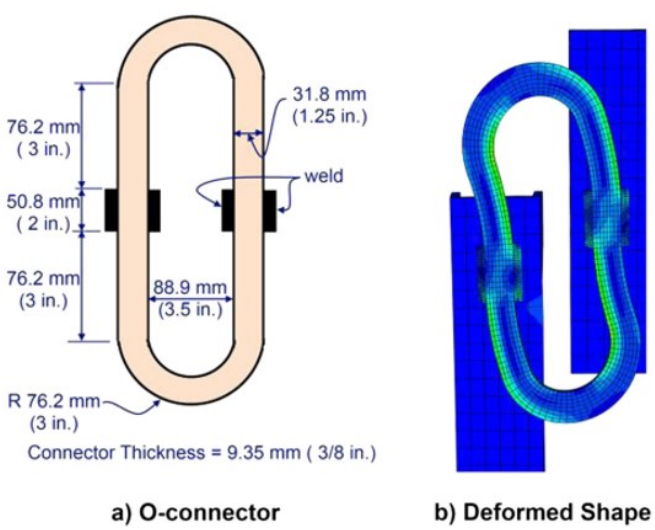



Figure 16. the geometric view of the O-connector and its deformed shape [31]

The wall and end columns can rock individually at the base when the lateral seismic loads were applied. The post-tensioned tendons are designed to remain elastic for the expected lateral loads up to the design-level earthquakes or a maximum expected lateral drift ratio. The function of post-tensioned tendons is the same as in typical unbonded post-tensioned precast concrete shear walls as introduced previously. The columns places at both sides of the wall aim to install replaceable connectors (O-connector) between the wall and columns.

According to Sritharan et al. [31], the initial stress in post-tensioning steel, $f_{pi}$, for the tendons in the wall is 174.4 ksi; for the tendons in the two end columns is 172.2 ksi and 176.2 ksi, respectively. The area of post-tensioning steel, $A_{pt}$, is 2.604 $in^2$ for the tendons in the wall; is 0.651$in^2$ for the tendons in the end columns. The post-tensioning tendons in the wall panel were stressed to an initial stress of 0.67 $f_{pu}$ at the beginning of the test, resulting in a total prestress force of 2019 KN (454.2 kips) acting at the center of the wall. The corresponding stress in the concrete wall was 1.05 ksi (7.25 Mpa), which was 18% of the specified concrete strength and 11.5% of the measured concrete strength on the day of testing. The prestressing steel in the north and south column was at a stress level of 0.66fpu and 0.68fpu after losses, anchoring the columns to the foundations with a force of 498.3 kN (112.1 kips) and 509.9 kN (114.7 kips), respectively.

**Comparison between PreWEC and Cast-in Place Shear Wall**

Sritharan et al. [31] did a test to compare the seismic response of PreWEC and cast-in-place shear wall under similar size, as shown in table 4.

Table 4. Dimension of PreWEC and Cast-in place shear wall.[31]

| | PreWEC | |
|---|---|---|

| | Panel | End column | Cast-in-place shear wall |
|---|---|---|---|
| Dimension | 5860 mm (height)<br>1830 mm (width)<br>155 mm (thickness) | 5630 mm (height)<br>203.2 mm (width)<br>152.4 mm (thickness) | 6400 mm (height)<br>2280 mm(width)<br>150 mm (thickness) |

The result shows that the performance of PreWEC performed better than the cast-in-place shear wall. After the cyclic loading, PreWEC performed exceptionally well with negligible damage to the wall panel and no damage to the end column under a 3% drift ratio. The damage to the wall was limited to spalling of cover concrete in the bottom corners. However, the cast-in-place concrete shear wall presents harsh damage compares to PreWEC under a 2.5% drift ratio. Figure 17 show the comparison of cracks between PreWEC and cast in place shear wall. The measured residual drift ratios of PreWEC were 0.35%, 0.45%, and 0.61% after subjected to ±2%, ±2.5%, and ±3%, which proves that the PreWEC has a good recentering ability. The PreWEC did not recent 100% because of the inelastic deformations of O-connectors.

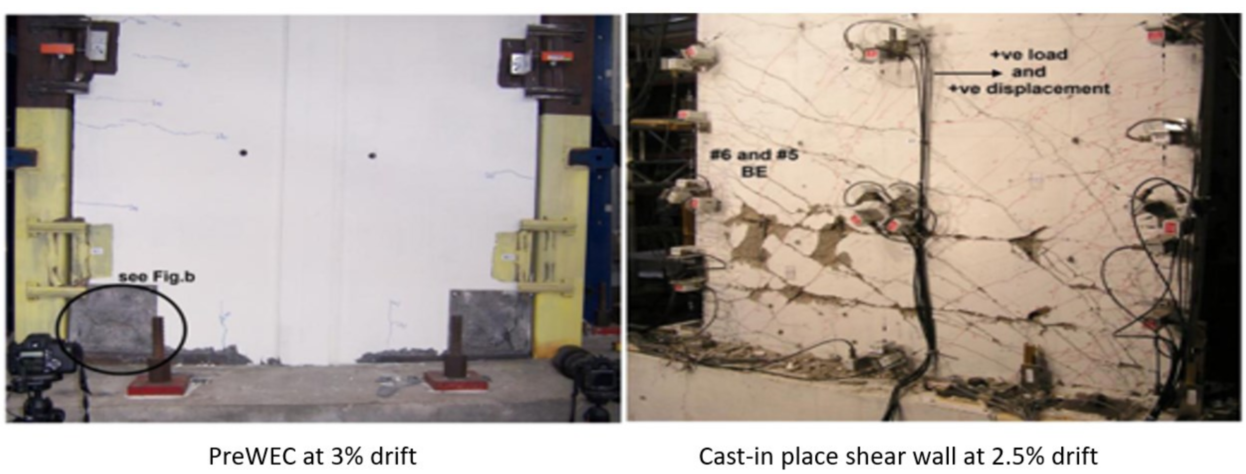



Figure 17. the cracks comparison between PreWEC and cast in place shear wall [31]

From the average response envelop of PreWEC and cast in place shear wall, the response of PreWEC performed better than cast in place shear wall by 38% of elastic stiffness and 12-17% lateral load resistance at any given lateral displacement. As figure 18 shows. For example, the lateral load value of PreWEC at a 2% lateral drift ratio was 506 KN, which was 13% greater than the target value established from the response of the cast-in-place shear wall.

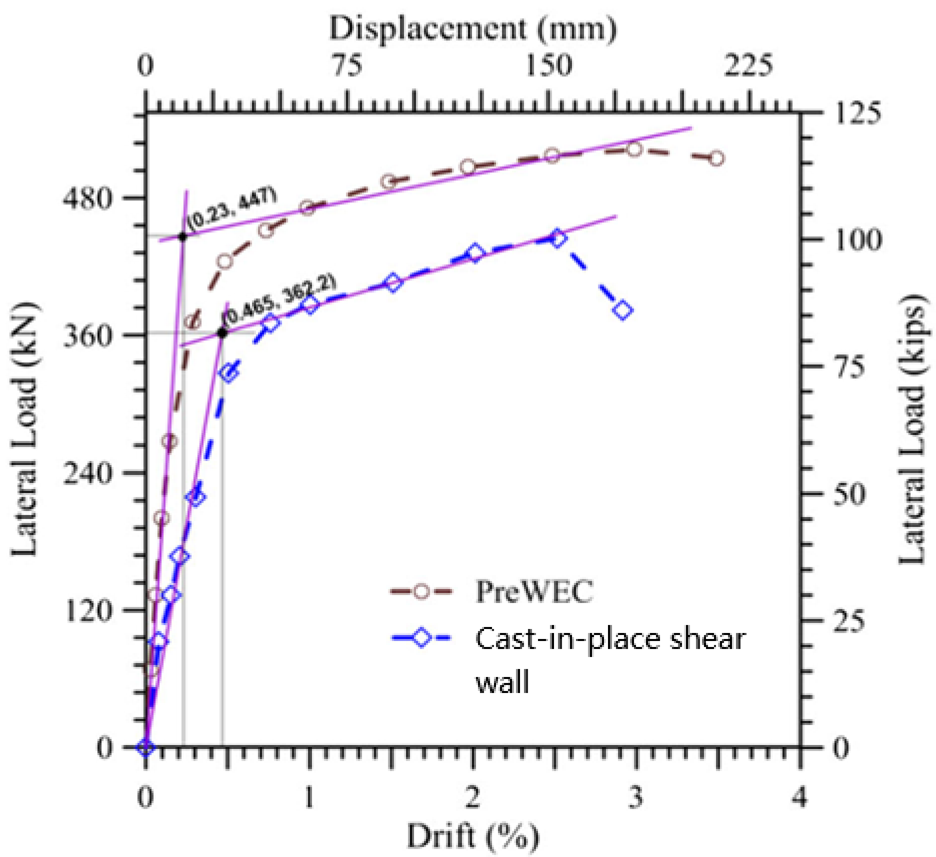



Figure 18. Response envelopes of PreWEC and cast-in place shear wall [31].

**Comparison of Energy Dissipating Ability of Unbonded Post-Tensioned Shear Wall with and without End Columns**

Twigden et al. [32] did a comparative experiment of two unbonded post-tensioned shear walls under cyclic load, one with end columns and another without end columns. The PreWEC still used O-connectors at the vertical joint between the shear wall and end columns. The cross-section of the walls is shown in figure 19.

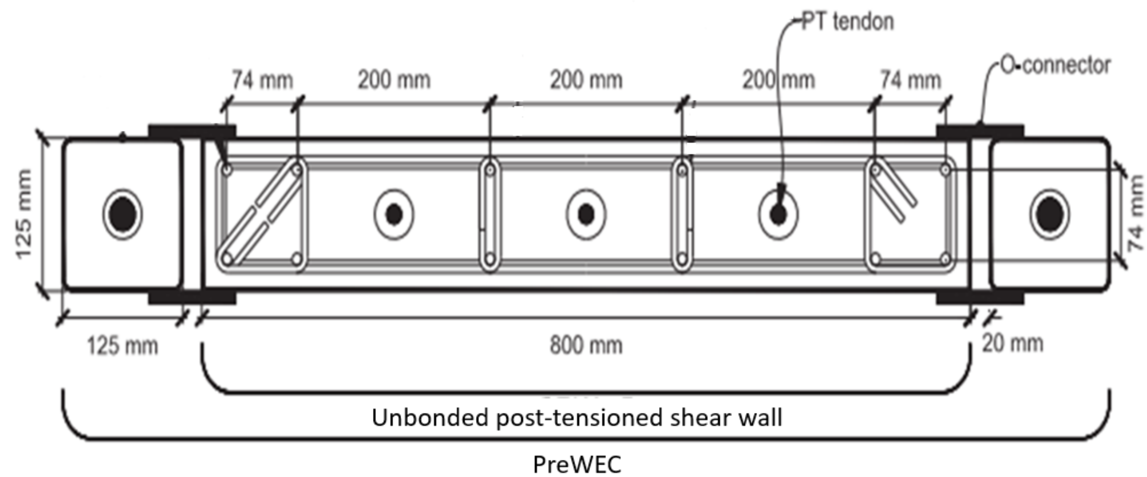



Figure 19. cross section of unbonded post-tension shear wall and PreWEC [32].

The measured lateral force-displacement response of each test wall was shown in figure 20. The response indicates that the PreWEC has increased strength and hysteresis area due to the addition of the O-connectors compare to the wall without end columns. According to Twigden et al. [32], the PreWEC arrangement results in connector forces imposed on the wall panel that is equal and opposite. As a result of these balanced connector forces, the wall panel behavior is independent of the number of O-connectors, and supplement damping can be added without compromising the wall design or performance.

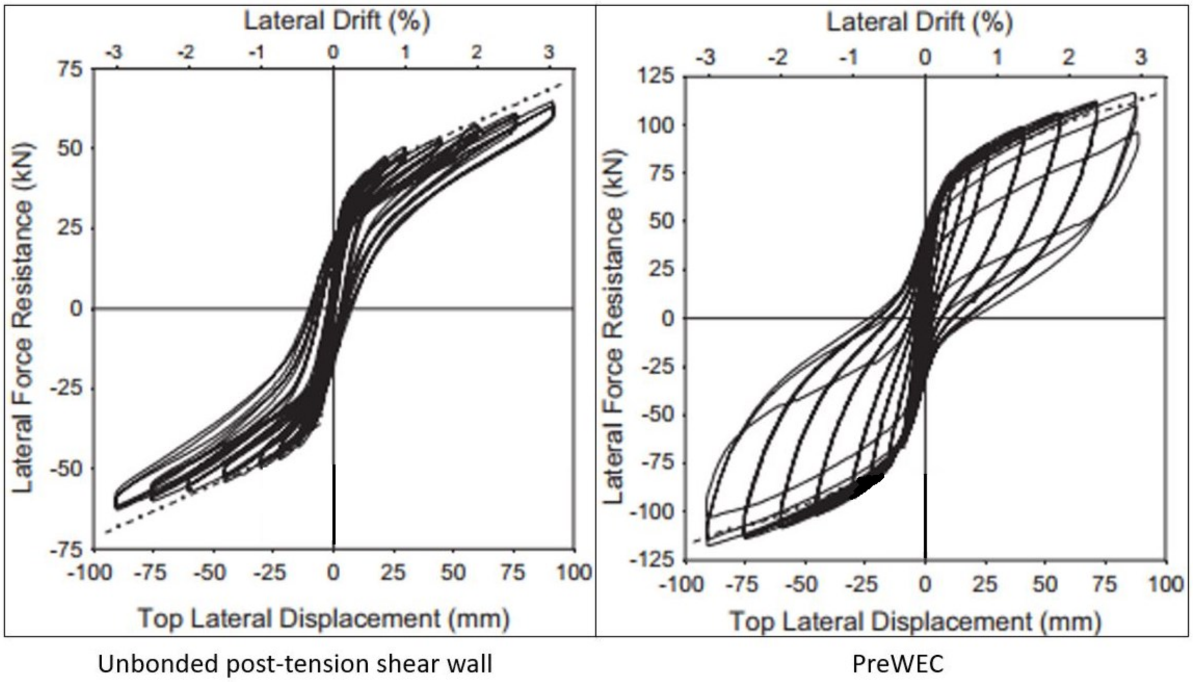



Figure 20. lateral force-displacement response of Unbonded post-tensioned shear wall and PreWEC [32]

# CHAPTER 4. ANALYSIS

## Cracks

The damage of concrete can significantly affect the stiffness of the shear wall system. The damage patterns of grout sleeve shear wall, laminated shear wall, PreWEC, and cast-in-place shear wall were shown in figure 21. In comparison, the PreWEC performed a minimum crack pattern after cyclic load.

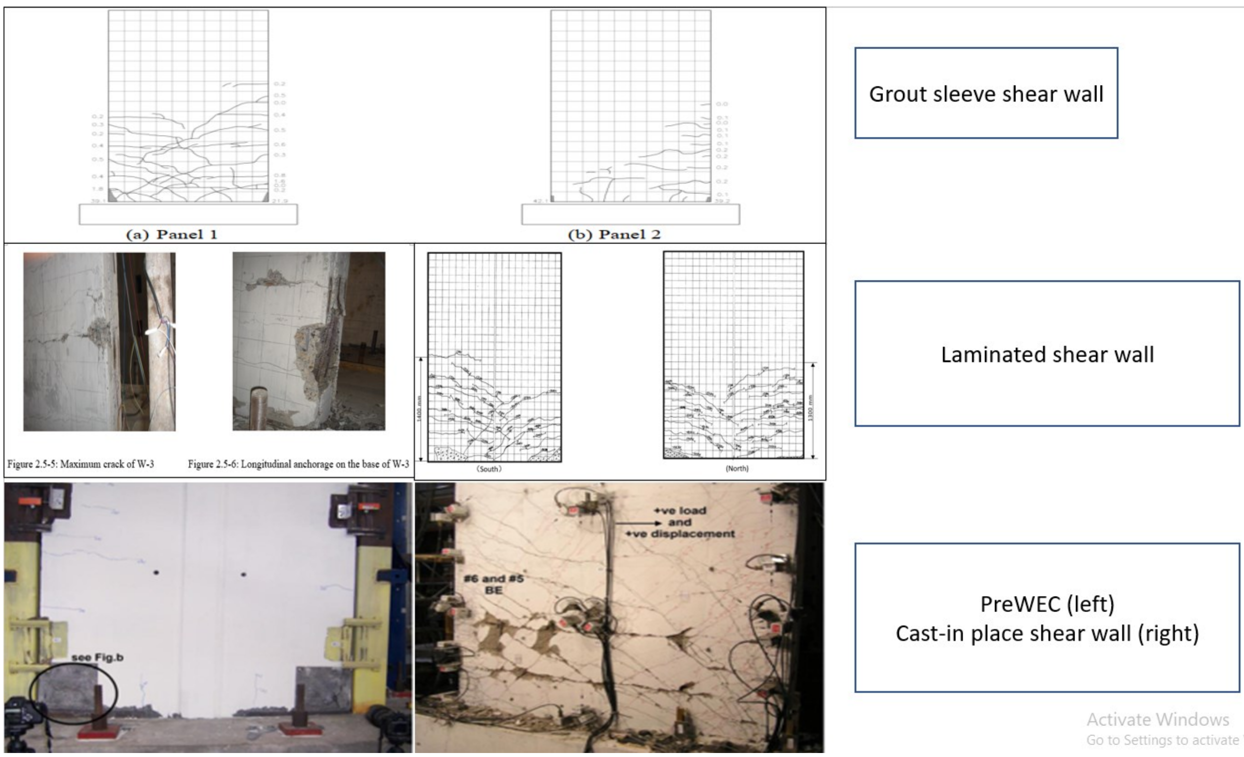


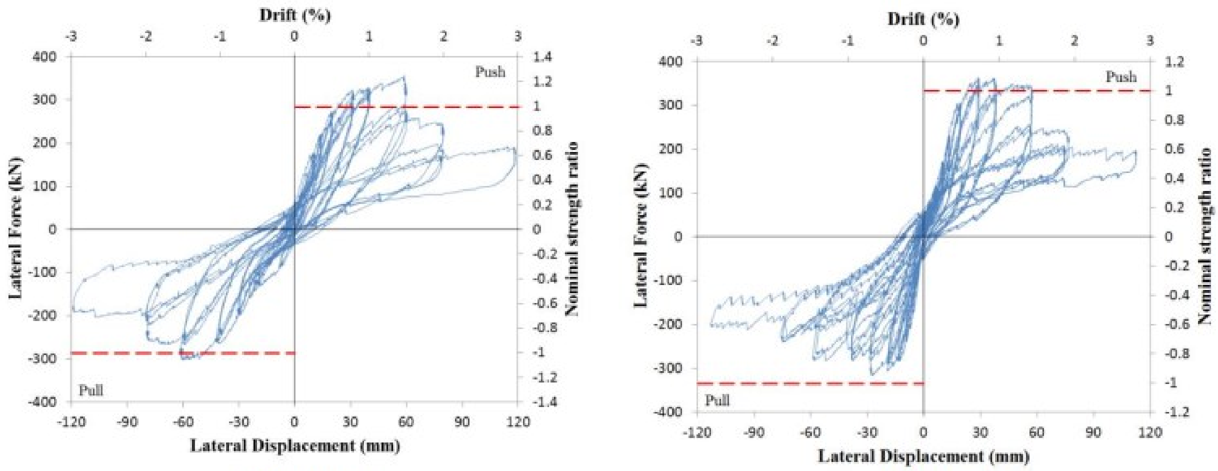

Figure 21. Comparison of the damage on shear wall, cast-in-place shear wall, laminated shear wall, unbonded post-tension shea wall, and PreWEC.

For the experiment of laminated shear wall tests, the shear walls were connected to the foundation by injecting longitudinal shear reinforcement. The concrete at the joint between the shear wall and foundation has different spalling degrees. The spalling area percentage to the whole wall surface area is 1.5% for cast-in-situ shear wall W1, 1.9% for laminated shear wall W2, 1.7% for laminated shear wall W3. From the grouted-sleeve concrete shear wall tests experiment, the concrete spalling area percentage to the whole wall surface area is 0.5% for the shear wall with a single reinforcement layer and 0.24% for the wall with a double-reinforced concrete layer. The reason for such differences is the different connection methods used between the shear walls and the foundation. It is possible to adopt the grout sleeve connecting strategy to laminated shear walls to reduce the concrete spalling, thus decrease the stiffness reduction. At the side faces of the laminated concrete shear wall, the concrete spalling attribute to the deformation of the reinforcement a lot. Using metal ducts with the proper size and grouting the longitudinal reinforcement inside the metal ducts may solve this problem. The metal duct between the reinforcement and the external concrete will protect the exterior concrete from the yield and deforming of the internal reinforcement, thus decrease inevitable cracks and concrete spalling. Therefore, it is possible to reduce the decreasing tendency of shear stiffness of the laminated shear wall.

## Cyclic behavior

### Grout sleeve shear wall

The analysis of the load-displacement curves under cyclic test for grout sleeve shear walls is as follows and shown in figure 22. Under the first few load cycles, the wall performed almost linear behavior. Then the yielding of the connection reinforcement leads to nonlinear behavior, and the hysteretic loop became flatter. In addition, the stiffness of the two sides of the load-displacement diagram performed differently due to the difference in thread slip at the two extreme grouted sleeve inserts after the almost linear behavior. Further, with an increase of the drift level, the diagrams pinching further because of furthering reinforcement yielding and the gap opening in the connection zone. Once the wall reached its maximum lateral force on both sides of the diagram, the wall will lose its lateral forces as 33% to 48% of its maximum values because of reinforcement pull-out and/or fracture. Reinforcement pull-out also leads to a degradation of wall stiffness, which was reflected in the slope of the diagram.




(a) Panel 1 (b) Panel 2

Figure 22. Hysteresis response for grouted shear wall panels

**Laminated shear wall**

Laminated shear wall W2 and W3 respond similarly to cast-in-situ shear wall W1. The failure characteristics are similar. The load-displacement curve of W1, W2, and W3 went through phases of linear, nonlinear, non-linear with a significant increase in displacement (pinching/elastoplastic phase), obviously non-linear (displacement increases faster but load increase slowly, plastic phase), destruction phase starts. The hysteresis curves of W3 and W2 are similar to W1; the difference between them is that the curve of W2 and W3 has a higher pinching effect or heavier elastoplastic phase than the curve of W1.

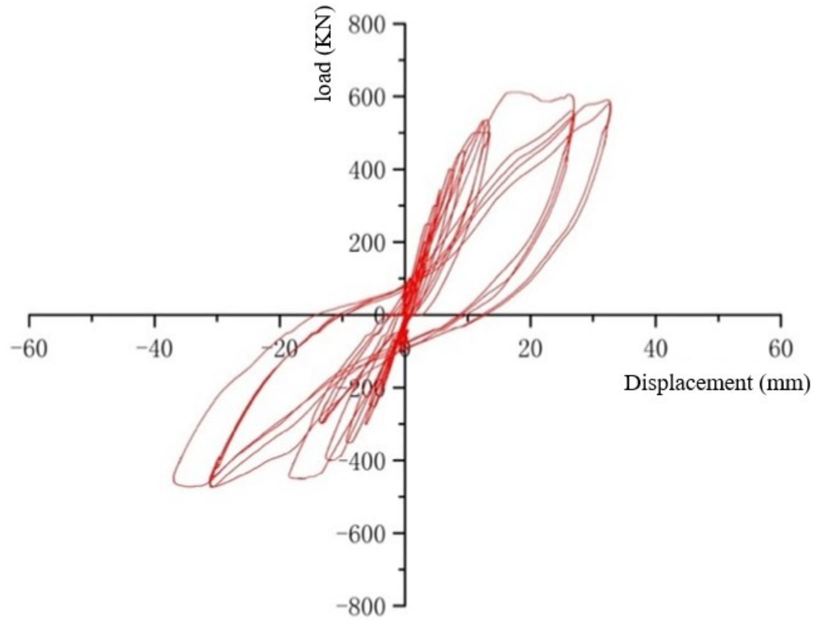



Figure 23. W-1 load displacement control curve [33]

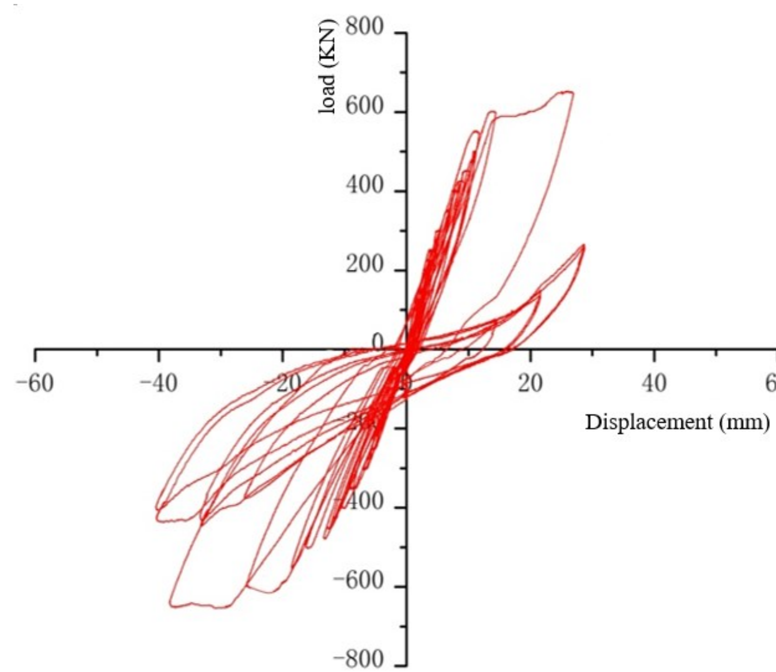



Figure 24. W-2 load displacement control curve [33]

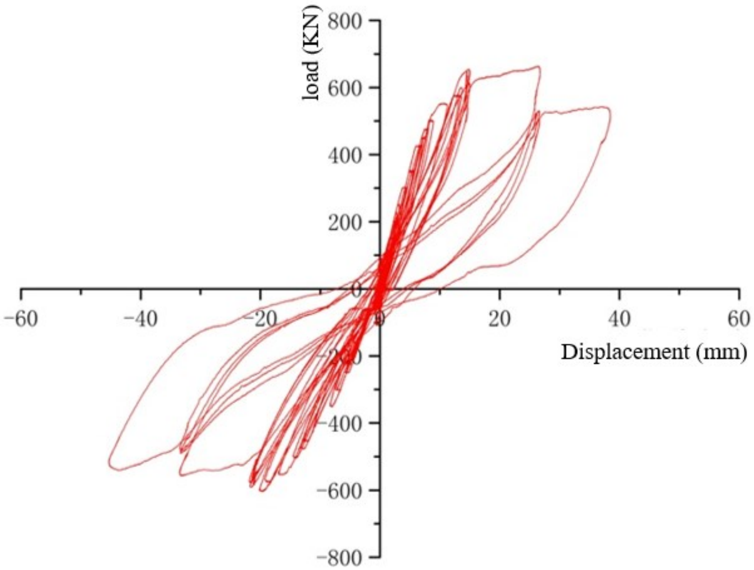



Figure 25. W-3 load displacement control curve [33]

**Unbonded post-tension shear wall**

The unbonded post-tension shear wall presented excellent self-centering ability under quasi-static loading up to a maximum drift of 2.5% from the lateral force-drift response shown in figure 26[30]. Before the gap happened at the horizontal joint, the wall presents a high initial stiffness and linear behavior. After gap opening, the wall behavior was nonlinear, and the stiffness began to decay due to both yielding of the unbonded post-tensioning tendons and gap opening at a 0.2% drift. With further lateral displacement, lateral stiffness decayed due to losses in the post-tensioning of tendons and the gap opening. With increasing drift level, the wall still presents a peak lateral load capacity of 40 kips. This phenomenon proves a stable horizontal force resistant ability of the wall. Most tendons lost their post-tensioning force during the load activity. The wall lost lateral stiffness indeed; however, the figure proved that the wall returns to its initial position, i.e., zero drift, fatherly, demonstrated its self-centering ability during the test.

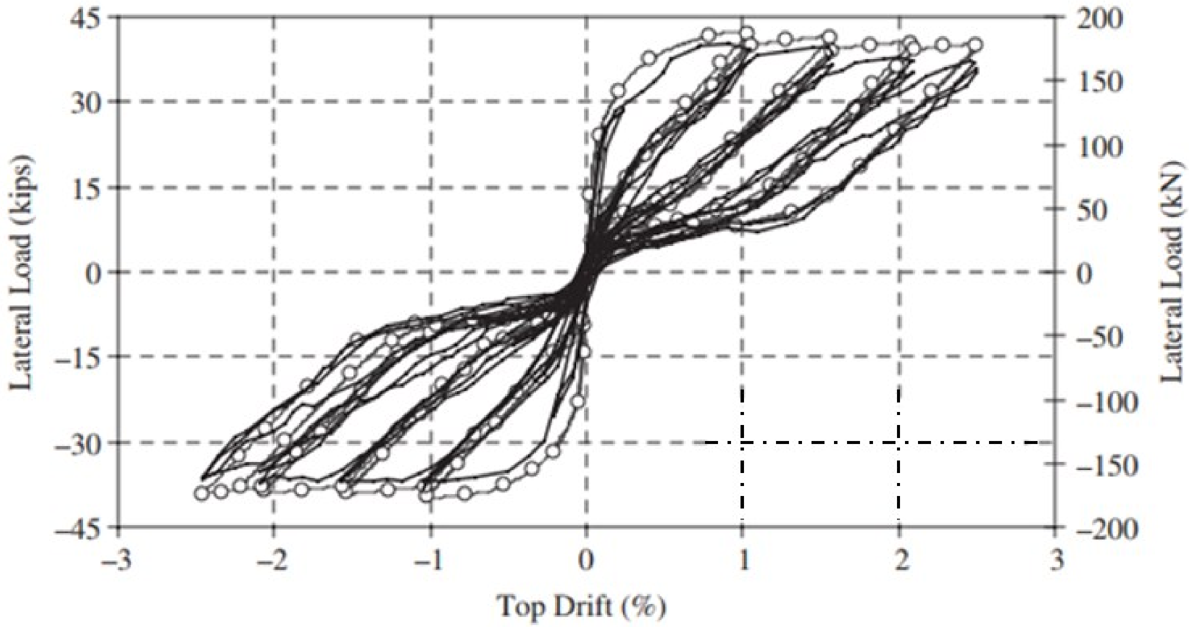



Figure 26. lateral load – top drift relationship of the unbonded post-tensioned shear wall sample [30].

**PreWEC**

The cyclic response of PreWEC proves a good self-centering ability. There is no significant strength degradation until the O-connectors started fracturing during the test. There is only minimal stiffness degradation. Fracture of the connections at a 3% drift level [32], as shown in figure 27. When the fracture of connections happened, the wall loses considerable lateral force resistance.

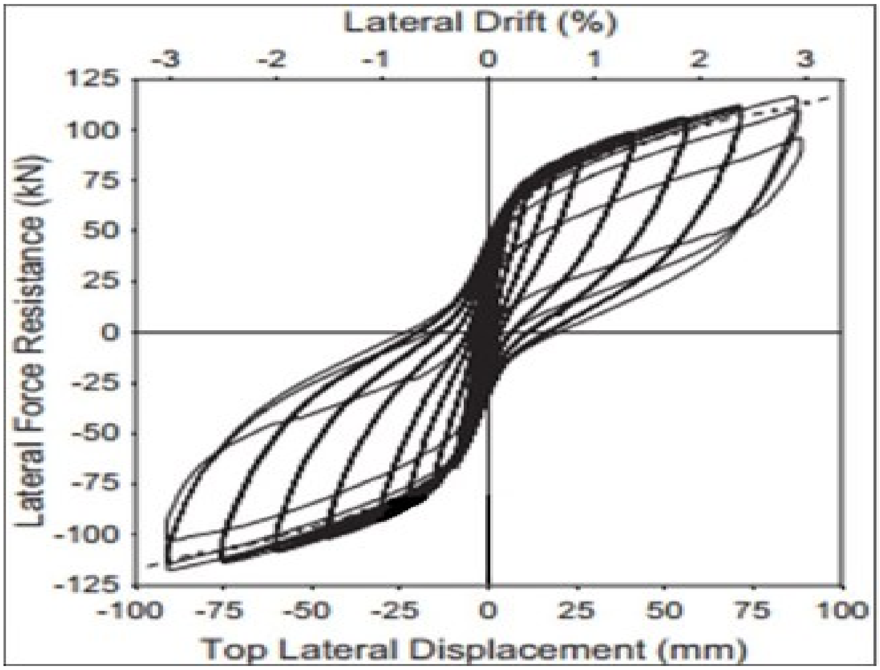



Figure 27. cyclic response of PreWEC [32].

**hysteretic energy dissipation capacity**

Larger hysteretic energy dissipation capacity provides a shear wall system with better seismic performance. The hysteretic loop of grouted sleeve shear walls, cast-in-place shear wall, laminated shear walls, unbonded post-tensioned shear wall, and PreWEC can be compared from figure 28. PreWEC performed best hysteretic loop.

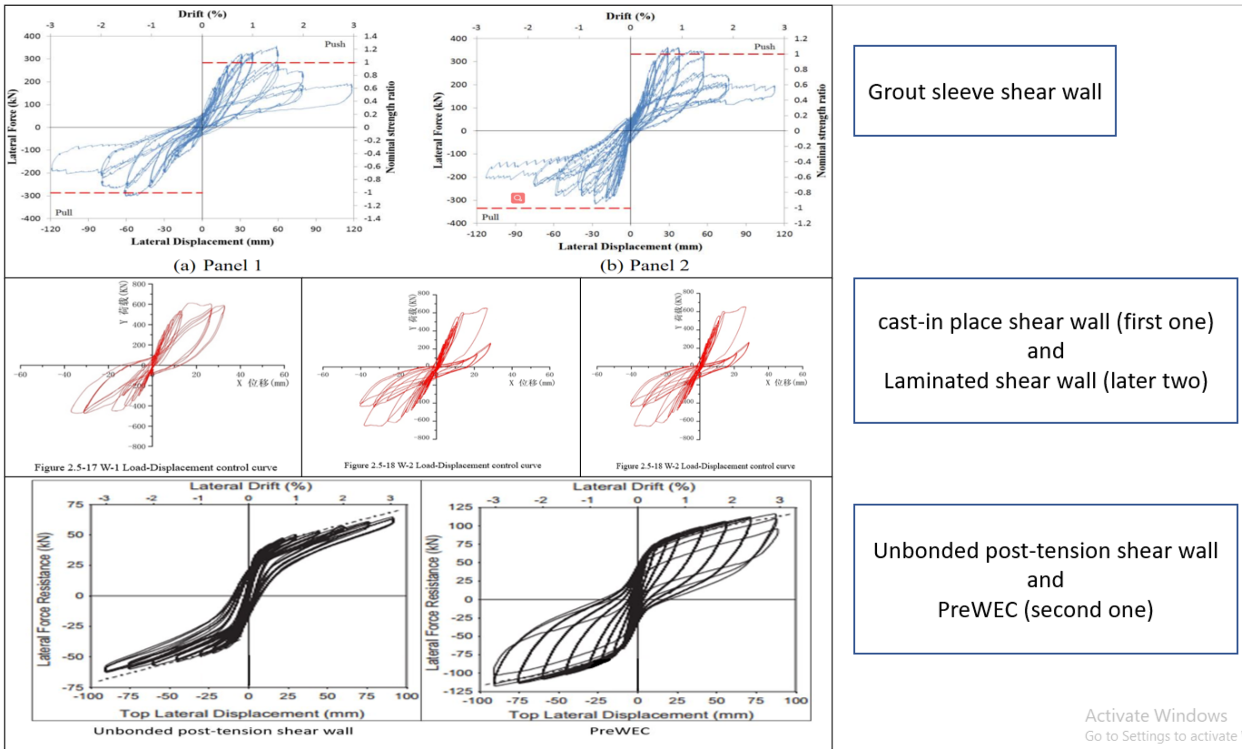



Figure 28. Comparison of hysteretic loop of grout sleeve shear wall, cast in place shear wall, laminated shear wall, unbonded post-tensioned shea wall, and PreWEC.

**Cost comparison**

The unit price of different materials refers to *Facilities Construction Costs with RS Means data (2020, 35th annual Edition).* Grouted sleeve shear walls are cheaper than others; however, their seismic performance is worse than others. The cost of PreWEC is higher and is 41% more than the cost of the corresponding cast-in-place shear wall with the same expected seismic response and size. The price of laminated shear walls is higher than that of related cast-in-place shear walls of the same expected seismic response and size. The details of the cost to different shear walls can check in the Appendix II.

The cost listed here is mainly the material fee. Although the precast shear wall cost was higher than that of the corresponding cast-in-place shear wall, the precast shear wall saves money from other aspects. The molds can be used hundreds of times before they are recycled and replaced. On-site concrete casting requires numerous workers, vehicles, and equipment, whereas precast concrete can save human resources. Many factors affect the quality of cast-in-place concrete, whereas precast concrete controls them due to indoor operation. High-quality construction, in another way, is an economic index.

Table 5. Cost of different shear walls. (Materials only).

| name | Price ($) |
|---|---|
| Grouted sleeve shear wall with single reinforcement layer | 392.37 |
| Grouted sleeve shear wall with double reinforcement layer | 451.34 |
| PreWEC | 2145.56 |
| Cast-in-place shear wall relates to PreWEC | 1518.76 |
| Laminated shear wall (rigid connector) | 773.08 |

| | |
|---|---|
| Laminated shear wall (simple connector) | 625.58 |
| Cast-in-place shear wall relates to laminated shear wall | 481.87 |

# CHAPTER 5. Conclusion

1. The crack patterns of grouted sleeve shear walls are better than that of laminated shear walls. By adopting the grouted sleeve connecting strategy to laminated slab concrete shear wall, the spalling of the concrete from laminated slab concrete shear wall would be reduced, thus protecting the stiffness reduction of it.
2. Grouted sleeve connection considerably affects the cyclic response of grouted sleeve shear wall. The thread slip effect results in a reduction of the wall panel stiffness. Reinforcement pull-out and fracture would lead to a significant lateral force loss.
3. Laminated shear wall simulates cast-in-place shear wall well in terms of similar failure characteristics, cyclic response, hysteresis curve.
4. Unbonded post-tensioned shear wall presents excellent self-centering ability under cyclic load. Oversized ducts between post-tensioned tendons and surrounding concrete avoid the bond stress transferring from the tendons to the surrounding concrete, thus avoid the damage that happened to the adjacent concrete. The horizontal joint provides an open-close free gap when subject to cyclic load.
5. PreWEC has increased strength and hysteresis area compare to the unbonded post-tension shear wall without end columns due to the adoption of O-connections.
6. PreWEC performed better than the corresponding cast-in-place shear wall in terms of damage development, elastic stiffness, and corresponding lateral load resistance at any different drift level.

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# APPENDIX I

## Details of Laminated Shear Wall

Table 6. details of reinforcement of W1, W2 and W3 of laminated share wall test [33]

| | W1 | W2 | W3 |
|---|---|---|---|
| details. | Cast-in-situ RC wall plate equipped with a Φ 12 @ 200 (HRB335) double-layer two-way reinforced mesh. | Vertical joints have Longitudinal reinforcement (HRB335) and stirrups (HPB235) in accordance with provisions of “JGJ3”.<br><br>(Rigid panel connection) | Equipped with horizontal connecting bars Φ 12 @ 200 (HRB335) in vertical joints in accordance with the provisions of “JGJ3”.<br><br>(With less reinforcement in the connection) |

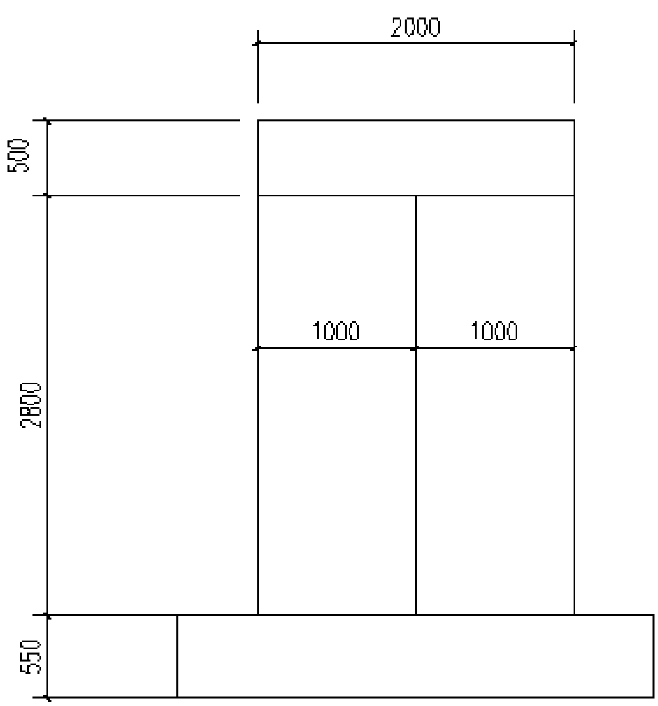
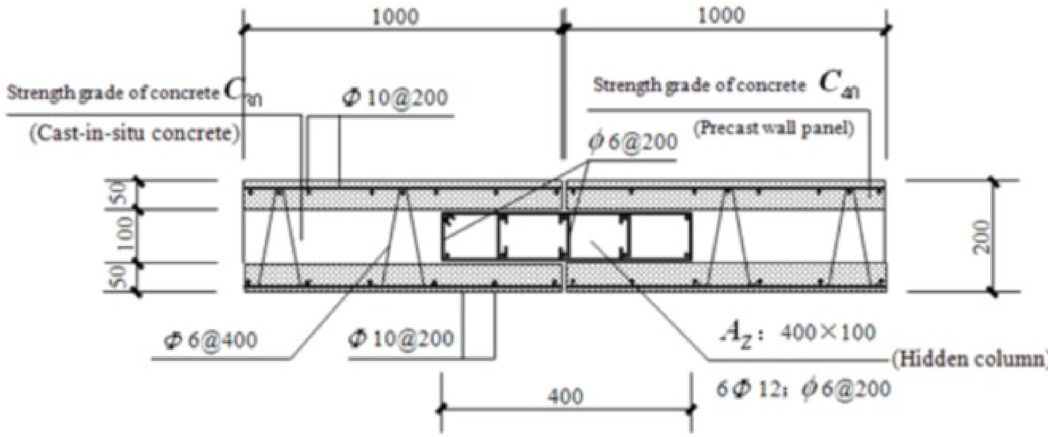
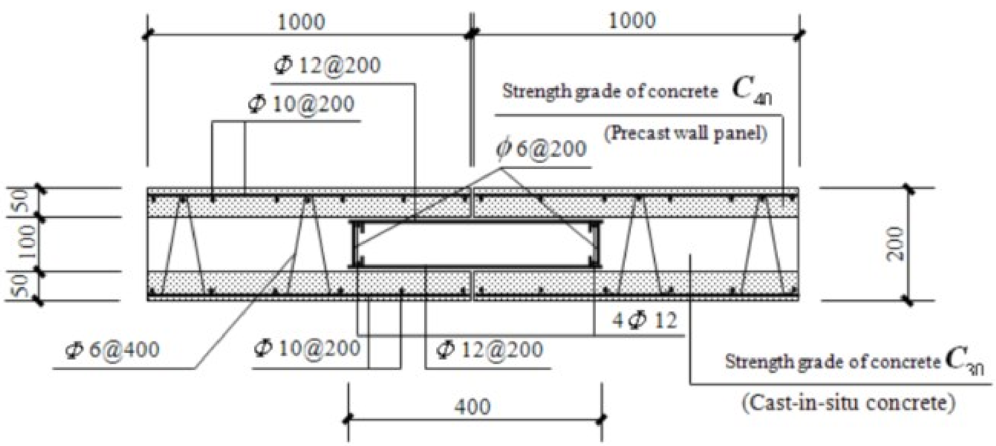



Figure 29. Details of W2 and W3 set up (unit in mm) [33]




a.plan view of W2

Figure 30. Top view of W2 wall plate (concealed column at the joint) [33]




b.plan view of W3

Figure 31. Top view of W-3 wall plate (horizontal ribs are set at the joint) [33]

## Appendix II

### Details of cost estimation.

Table 7. cost details of Grouted sleeve shear wall with single reinforcement layer

| Grouted sleeve shear wall with single reinforcement layer | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 1.57 cubic yard | 6000 psi concrete, ready Mix | 153/cubic yard | 240.21 |
| 32,000mm | grade 500 HD12 | 11.825 / 6000mm | 57.33 |
| 28,000mm (95.5344lb) | #5 bar | 34.33 / ton | 1.64 |
| 2100mm | HD 16 rebar | 3.751 / 1000mm | 7.88 |
| 82.67 inch | ducts | 12.38/LF | 85.29 |
| 0.0148 cubic foot | Grout | 1.58/CF | 0.02 |
| | | SUM | 392.37 |

Table 8. cost details of Grouted sleeve shear wall with double reinforcement layer

| Grouted sleeve shear wall with double reinforcement layer | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 2.09 cubic yard | 6000 psi concrete, ready Mix | 153/cubic yard | 240.21 |
| 64,000mm | grade 500 HD12 | 11.825 / 6000mm | 114.67 |
| 56,000mm (0.096ton) | #5 bar | 34.33 / ton | 3.28 |
| 2100mm | HD 16 rebar | 3.751 / 1000mm | 7.88 |
| 82.67 inch | ducts | 12.38/LF | 85.29 |
| 0.0148cubic foot | Grout | 1.58/CF | 0.02 |
| | | SUM | 451.34 |

Table 9. cost details of PreWEC.

| PreWEC | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 2.17 CY | Concrete | 153/cubic yard | 332.01 |
| 40ft | end column | 20/LF | 800.00 |
| 70320 mm (87.67lb) | #3 bar | 1.05/lb. | 92.05 |
| 70320 mm (154.5729lb) | #4 bar | 1.05/lb. | 162.30 |
| 371367.73mm (462.9856lb) | #3 hoops | 1.05/lb. | 485.10 |
| 42.62 lb. | post-tension tendons (2.6in^2) | 5.0/lb. | 213.10 |
| 12.20 lb. | post-tension tendons (0.5in^2) | 5.0/lb. | 61.00 |
| | | SUM | 2145.56 |

Table 10. Cost details of the cast-in-place shear wall relates to PreWEC

| cast in place shear wall relates to PreWEC | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 2.86 CY | Concrete | 153/cubic yard | 437.58 |
| 503.93ft (191.49lb) | #3 | 1.05/lb. | 201.06 |
| 167.98ft (112.55lb) | #4 | 1.05/lb. | 118.18 |
| 41.99ft (43.67lb) | #5 | 1.05/lb. | 45.85 |
| 83.98ft (125.97lb) | #6 | 1.05/lb. | 132.27 |
| 167.98ft (571.13lb) | #9 | 0.86/lb. | 491.17 |
| 519ft (88.23lb) | #2 hoops | 1.05/lb. | 92.64 |
| | | SUM | 1518.76 |

Table 11. Cost details of the laminated shear wall with rigid connection (W2)

| Laminated shear wall (with grid connection) | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 1.46 CY | Concrete | 153/cubic yard | 223.38 |
| 174.54ft (66.32lb) | #3 bar | 1.05/lb. | 69.64 |
| 477.68ft (320.04lb) | #4 bar | 1.05/lb. | 480.07 |
| | | SUM | 773.08 |

Table 12. Cost details of the laminated shear wall with simple connection (W3)

| Laminated shear wall (with simple connection) | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 1.46 CY | Concrete | 153/cubic yard | 223.38 |
| 31.23ft (11.86lbl) | #3 bar | 1.05/lb. | 12.45 |
| 551.17 ft (369.28lb) | #4 bar | 1.05/lb. | 387.74 |
| | | SUM | 623.58 |

Table 13. Cost details of the cast-in-place shear wall relates to laminated shear wall

| cast in place shear wall relates to laminated shear wall | Description | unit price. $ | Total. $ |
|---|---|---|---|
| 1.46 CY | Concrete | 153/cubic yard | 223.38 |
| 367.44ft (246.18lb) | #4 | 1.05/lb. | 258.49 |
| | | SUM | 481.87 |