INTERACTION BETWEEN ROCKING-WALL SYSTEM AND SURROUNDING STRUCTURE

Q.Z. Liu[1], C.W. French[2], S. Sritharan[3]

[1] *Research Scientist, Englekirk Structural Engineers, andyliu.us.se@gmail.com*
[2] *Distinguished Professor, University of Minnesota, Twin Cities, cfrench@umn.edu*
[2] *Wilkinson Chair Professor, Iowa State University, sri@iastate.edu*

Abstract

Rocking-wall systems are innovative structural systems developed to reduce damage and residual drifts as observed in buildings with regular structural walls. Key factors to implement the systems in buildings include developing reliable wall-floor connections and understanding the interaction between rocking-wall systems and surrounding structures. Two structural assemblages were tested under quasi-static cyclic loading with drifts up to 5%. A PreWEC (Precast Wall with End Columns) system, which consisted of a precast rocking-wall, energy-dissipating elements “O-connectors,” and adjacent end columns, was used as the rocking-wall system in the two assemblages. The surrounding structure in the first assemblage (PFS1) included a cast-in-place (CIP) unbonded post-tensioned slab with rigid wall-floor connections and CIP edge beam/columns. Test results of PFS1 showed that little damage occurred to the wall through 5% drift. Local damage occurred to the slab at 4% drift, but structural integrity of the slab remained due to the intact prestressed strands. Compared to the PreWEC system, strength of the first assemblage (PFS1) was greatly increased due to the resistance provided by the surrounding structure. Despite of the plasticity developed in the surrounding structure, PFS1 demonstrated reasonable self-centering behavior. The surrounding structure in the second assemblage (PFS2) included a precast slab with vertical-isolation wall-floor connectors and precast edge beam/columns. Test results of PFS2 showed that the slab was successfully isolated from uplift of the wall through the special wall-floor connectors. Compared to the PreWEC system, strength of the second assemblage (PFS2) was barely increased. The entire assemblage was almost damage-free at 2% design drift and demonstrated great self-centering behavior through 5% drift. Overall performance of the two assemblages was compared. Some challenges encountered for implementing the rocking-wall system in design practice are discussed.

Keywords: Experimental tests; Rocking-wall; Wall-Floor Interaction; Energy Dissipation; Residual Drift

1. Introduction

Special structural walls are popular lateral-force-resisting systems used in buildings in seismic regions. Generally, the walls are detailed to develop a plastic hinge region near the base and dissipate energy through inelastic response. Under this circumstance, structural engineers are able to design for a lateral force lower than the expected if the wall remained elastic. This design methodology provides benefit to reduce initial cost of the structure, but the energy-dissipation is associated with permanent damage and residual drifts of the structure in a large earthquake, causing tremendous economic losses due to repair/reconstruction cost and business downtime. These losses underscore the need to develop innovative structural concepts, such as base-isolated structures, rocking-wall systems etc.

Fig. 1 shows a comparison of rocking walls with special structural walls. When special structural walls are laterally displaced, tensile strain is developed in the concrete and the bonded rebar due to fixity at the base. The concrete cracks and the rebar eventually yield, forming a “plastic hinge region.” Different from special structural walls, rocking walls are clamped to the foundation by unbonded post-tensioned (PT) strands and the rebar in the walls are not continuous across the wall-foundation interface. When the walls are laterally displaced, they rock about the corners, as shown in Fig. 1. Therefore, the walls experience no cracking but a concentrated gap opening at the base. The PT strands contribute to the strength of the walls and provide restoring forces to the walls for self-centering.

Because the PT strands are recommended to remain elastic at design drifts [1], the energy-dissipation capacity of the wall alone is not large. Generally, supplementary energy-dissipation elements are added to the walls. A rocking-wall system named PreWEC (Precast Wall with End Columns), as shown in Fig. 2, was introduced by Sritharan et al. [2]. Two end columns are placed next to the rocking wall and oval-shaped “O-connectors” serve as energy-dissipation elements. When the wall rocks, relative vertical displacements exist between the wall and the end columns. The O-connectors are attached to the wall and the end columns, thus they are deformed and energy is dissipated through hysteretic damping. A half-scale PreWEC system (6-m high) was successfully tested and the system demonstrated excellent energy-dissipation and self-centering behavior [3]. The PreWEC system was used in the test assemblages discussed in this paper.

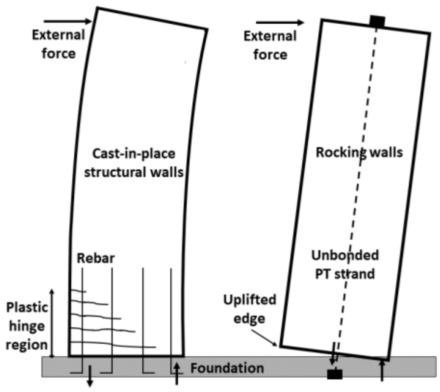

Fig. 1 Rocking wall and special structural wall [4]

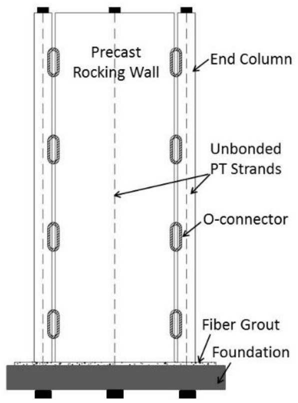

Fig. 2 Schematic view of PreWEC [5]

Performance of an isolated rocking-wall system has been studied extensively, but there are only a few tests that include both rocking-wall systems and surrounding structures to study its application in buildings [6,7,8]. Due to complexity of these tested structures (e.g., full-scale buildings with multiple stories and spans), quantitative studies of their performance, especially the interaction between rocking-wall systems and surrounding structures, are difficult to discern. In this study, two one-third scale rocking-wall assemblages were tested at the MAST (Multi-Axial Subassemblage Testing laboratory, University of Minnesota, Twin Cities). The two test assemblages were designed based on the same prototype building, which was a six-story office building located in the State of California. A single frame line and the first story of the building was selected for testing, while other frames and stories of the building was emulated by introducing some boundary conditions [4,5]. In the following, a brief description of the two tests is presented first, followed by comparsion of the performance of the two assemblages.

2. First test assemblage PFS1

2.1 Design of PFS1

The first test assemblage was designed to maximize the potential interaction between rocking-wall systems and surrounding structures by using rigid wall-floor connections and CIP surrounding structures. Fig. 3 shows an overview of PFS1.

The PreWEC system used in PFS1 was of an alternate style compared to that shown in Fig. 2. As shown in Fig. 3, rather than being next to the ends of the wall, four end columns, which were made of steel tubes, were placed on the front and the back surfaces near the ends of the wall. The precast wall panel was 2286 mm long, 5690 mm tall and 152 mm thick. It was clamped to the base block by five 13 mm diameter 7-wire GR270 strands with an initial prestressing force approximate 500 kN. Two O-connectors were welded to each tube and the embedded plates in the wall, and there were eight in total in the PreWEC system.

A CIP unbonded PT slab and two CIP edge beam-columns were constructed as the surrounding structure. Fig. 4 shows the connection between the wall and the slab. As shown in the figure, four #3 (9.5 mm diameter) rebar were distributed along the wall-slab interface as shear-friction dowel to transfer in-plane forces from the slab to the wall; two rebar and two unbonded PT strands in the slab went through each wall end to maintain structural integrity of the slab. After pouring the concrete, it was expected that a rigid wall-floor connection was formed and the gravity loads from tributary floor areas of the wall were directly transferred to the wall.

Quasi-static cyclic loading was applied to the test assemblage using the MAST crosshead system, which had the capability of controling six degrees of freedom. Primary control of the loading in this test was the lateral displacement at the top of the top block. Three cycles of displacements at each drift level were applied until 5% drift, which was limited by the maximum available stroke of the actuators. A constant axial load was applied to the wall through the crosshead, simulating the gravity loads sustained by the wall.

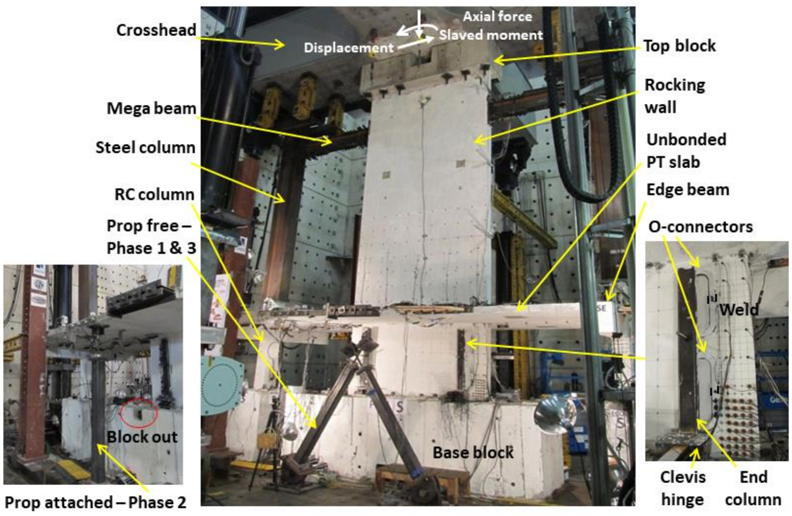

Fig. 3-Overview of PFS1 [4]

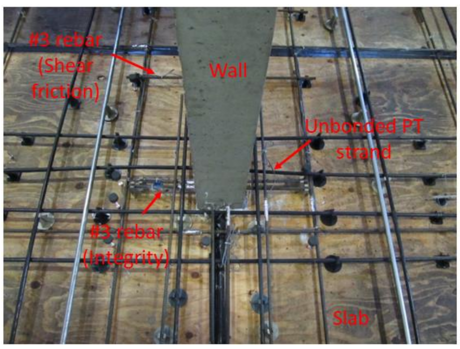

Fig. 4-Wall-floor connection in PFS1

2.2 Test results

Fig. 5 shows the force-displacement response of PFS1 in the test. Detailed loading protocol in different phases is described in the literature [4]. The flag-shaped hysteretic curves shown in the figure demonstrate its great energy dissipation and self-centering behavior.

The stiffness of the assemblage started to decrease at 0.25% drift, indicating that the wall had uplifted at this drift level. The strength of the assemblage was 414 kN at 0.25% drift, and it increased to 614 kN at 2% design drift and reached the maximum (627 kN) at 2.5% drift. The residual drift of the assemblage was only 0.58% at the end of the 2.5% drift cycles. O-connectors started to fracture when the assemblage was loaded towards 4% drift in Phase 3, which caused a drop in the strength (point D and E in the figure). The strength of the assemblage dropped to 498 kN at 5% drift, which was 80% of its peak strength. The residual drift of the assemblage reached the peak (0.76%) after 4% drift cycles, which was small compared to tranditional special structural walls.

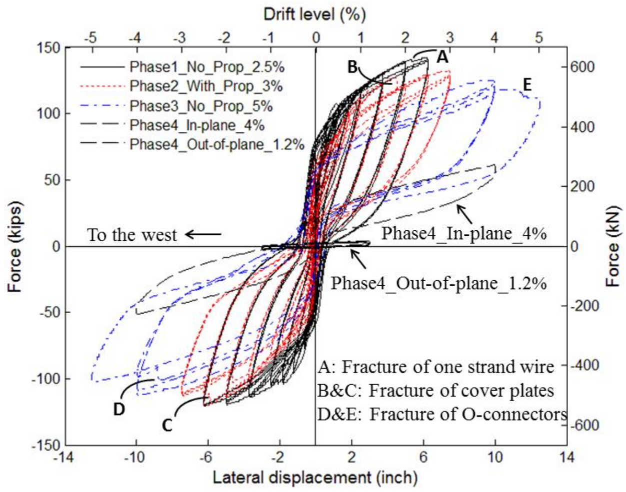

Fig. 5-Force-displacement curves of PFS1 [4]

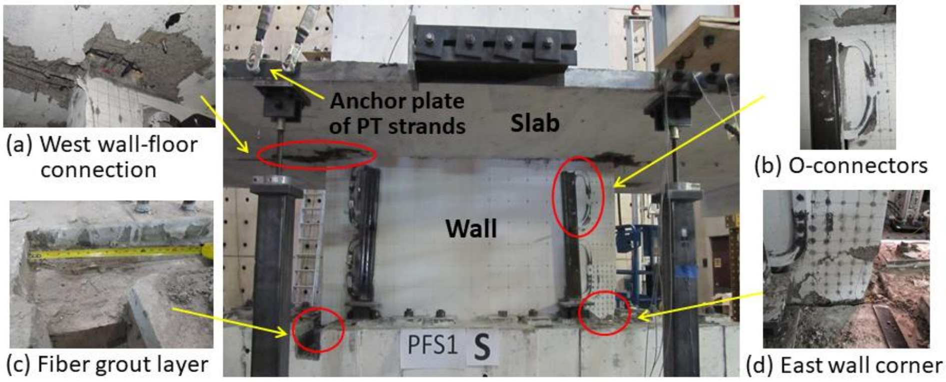

Fig. 6- Damage to the first story of PFS1 [4]

Fig. 6 shows an overview for the first story of PFS1 at the end of the test. As shown in Fig. 6, little damage occurred to the wall panel throughout the test and the wall was deemed reusable after the test. The PT strands in the wall started to yield when the assemblage was loaded towards 3% drift. The residual PT force in the wall was approximately 48% of the initial PT force after the 5% drift. Yielding of the O-connectors occurred at 0.5% drift. Fracture of the O-connector occurred at the weld of the O-connectors during 4% drift cycles. Damage to an O-connector at the end of the test is shown in Fig. (b). The O-connector was distorted from an oval shape closer to a round shape.

Cracks in the slab started to form at the floor-edge beam connections and adjacent to the wall ends during the 0.25% and 0.5% drift cycles, respectively. Yielding of rebar occurred at the wall-floor connection and the floor-edge beam connection at 0.75% and 1% drift, respectively. When the assemblage was displaced to larger drifts, concrete spalling and rebar buckling occurred at the wall-floor connections at 4% drift, as shown in Fig. 6(a). The damage to the slab was localized, and structural integrity of the entire slab was maintained due to the intact PT strands in the slab. The PT strands did not yield throughout the test because they were lightly prestressed initially. Most of the cracks in the slab closed during unloading because of the prestressing forces in the slab. The slab would be reusable after the local damage was repaired.

3. Second test assemblage PFS2

3.1 Design of PFS2

The second test assemblage was designed to minimize the potential interaction between rocking-wall systems and surrounding structures by using vertical-isolation wall-floor connectors and precast surrounding structures. Fig. 7 shows an overview of PFS2.

The PreWEC system in PFS2 was of the same style shown in Fig. 2. Two 254 mm long and 152 mm thick precast RC end columns, in which the steel confinement was designed based on Section 21.6 of ACI 318 [9], were placed 25 mm next to the ends of the wall. The precast wall panel was 1727 mm long, rendering the footprint of the PreWEC system in PFS2 the same as that in PFS1 (2286 mm). To make the strength of the PreWEC system in PFS2 similar to that in PFS1, the wall was clamped to the base block by seven 13 mm diameter 7-wire GR270 strands with an initial prestressing force approximate 790 kN. Four O-connectors were welded to the embedded plates in the wall and in each end column, and there were also eight O-connectors in total in PFS2.

A precast slab and two precast edge beam-columns were constructed as the surrounding structure. Fig. 8 shows an overview of the slab. The slab was formed by assembling twelve pieces of precast planks. Mini V connectors were used as chord and web connections between the planks. Steel angles attached to the transverse/edge beams were used to support the planks on the bottom, enabling the gravity loads from the slab being transferred to the end columns/edge columns. Steel strap plates were used across the plank-edge beam and plank-transverse beam interface, connecting the planks together on the top.

Special vertical-isolation wall-floor connectors, named as V-connectors [10], were used in PFS2. As shown in Fig. 9, the connector consisted of a slotted insert embedded in the wall and a V connector affixed to the floor. A bolt fed into a nut inside the slotted insert (not shown in the figure), connecting the plate to the insert. Because the plate was able to vertically slide along the insert, no gravity loads but lateral forces from the slab were transferred to the wall through the special wall-floor connectors.

Similar to PFS1, quasi-static cyclic loading was applied to the test assemblage with primary control being the lateral displacement at the top of the top block. Different from PFS1, there were no axial loads applied to the wall because of the vertical-isolation wall-floor connectors. To emulate the gravity loads from the precast slab, which were sustained by the columns in PFS2, threaded rods were placed inside each end column/edge column and post-tensioned.

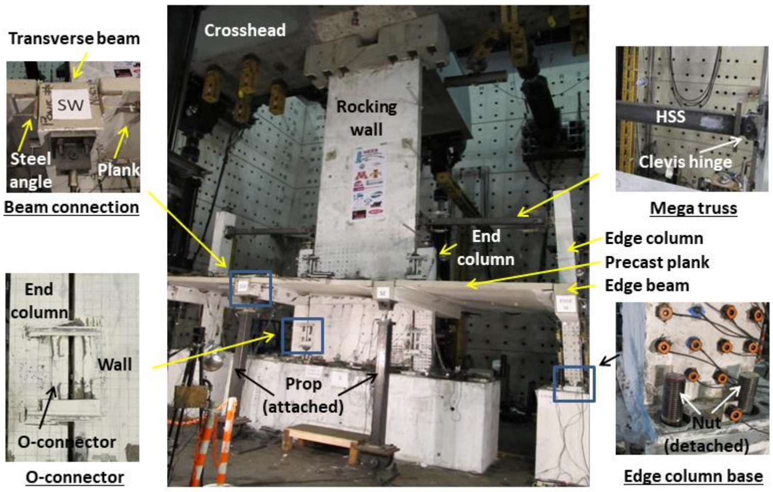

Fig. 7-Overview of PFS2 [5]

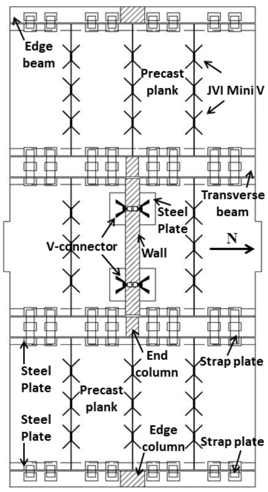

Fig. 8-Overview of slab

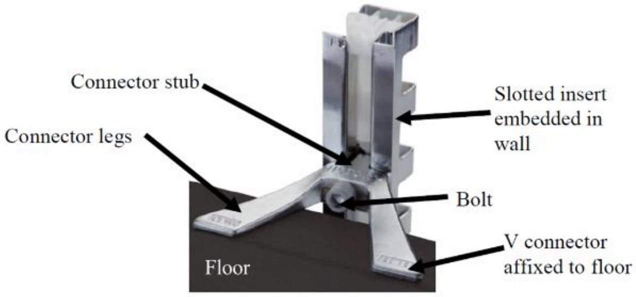

Fig. 9-Wall-floor connectors in PFS2 [10]

3.2 Test results

Fig. 10 shows the force-displacement response of PFS2 in the test. Detailed loading protocol in different phases is described in the literature [11]. Similar to PFS1, the flag-shaped hysteretic curves shown in the figure demonstrate its great seismi performance.

The strength of the assemblage was 258 kN at 2% design drift, and it reached the peak (294 kN) at 4% drift. A numerical model of the isolated PreWEC system used in the assemblage, which included the wall, the end columns and the O-connectors but excluded surrounding structures, was established using ABAQUS [12]. The pushover curve obtained from the numerical simulation is superimposed in Fig. 10. As shown in the figure, the strength of the test assemblage only slightly exceeds that of the simulated PreWEC system. It was because the slab was successfully isolated from uplift of the wall, the surrounding structures had little impact on the strength of the test assemblage. As shown in Fig. 10, the residual drifts of the assemblage were negligible throughout the test. The peak residual drift was only 0.15% recorded after the 4% drift cycles.

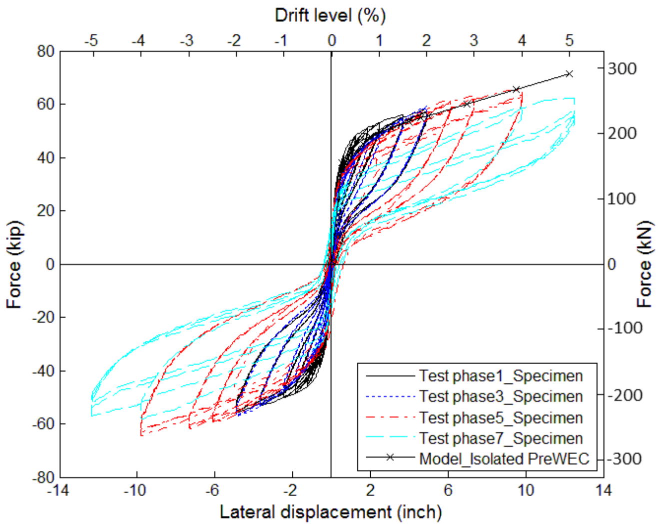

Fig. 10-Force-displacement curves of PFS2 [5]

Fig. 11 shows an overview for the first story of PFS2 at the end of the test. Similar to PFS1, little damage occurred to the wall throughout the test, and the wall was deemed reusable after minor repair. Yielding of the O-connectors occurred at 0.75% drift, and fracture of the O-connectors at the weld was observed during 4% drift cycles. Some O-connectors only failed partially at the end of the test. As shown in Fig. 11(b), half of the O-connector fractured while the other half stayed connected to the wall and the end column (U-shaped portion at the bottom). The "U-shaped" O-connector was still capable of dissipating energy when it was cyclically deformed. Spalling of concrete cover started to occur at both end columns during 1.5% drift cycles, but the damage was minor. The damage to the end columns increased at 4% drift. Although the axial force demand on the end columns was large due to the gravity loads from tributary floor area of the wall, the damage was mainly caused by poor concrete consolidation during fabrication and column base detail [11].

As shown in Fig. 11(a), little damage occurred to the precast slab throughout the test. As expected, the planks, the Mini V connections and the V-connectors remained intact at the end of the test. Yielding of the steel strap plates at the plank-beam connections did not occur until the 4% drift cycles. Although the planks were isolated from uplift of the wall, kinking of the strap plates occurred at the plank-beam connections due to the relative rotation between the planks and the end columns/edge columns. None of the strap plates fractured at the end of the test, and the untopped precast slab was deemed reusable after the test.

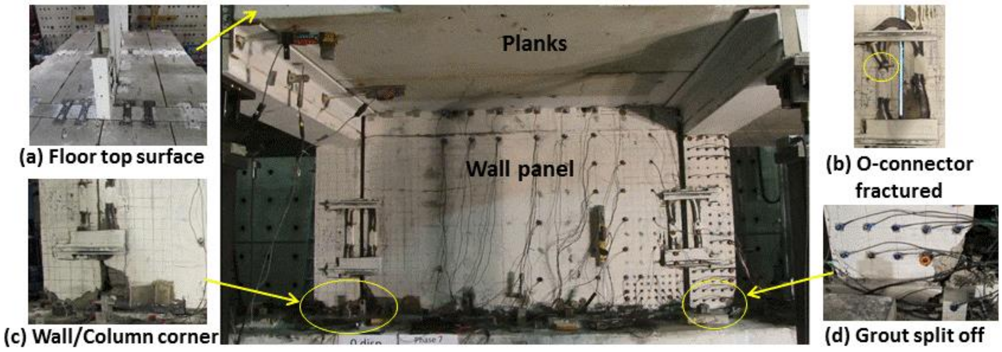

Fig. 11- Damage to the first story of PFS2 [5]

4. Comparison of PFS1 and PFS2

As mentioned earlier, the isolated PreWEC system in the two assemblages were designed to be equivalent, thus a direct comparison of the performance of rocking-wall structures using different types of wall-floor connections was made possible.

As shown in Fig. 6 and Fig. 11, little damage occurred to the precast rocking-wall panel in both tests. The steel end columns in PFS1 remained elastic, but some concrete spalling occurred to the RC end columns in PFS2. As expected, the energy-dissipation elements (O-connectors) yielded and fractured in both tests. Overall, the PreWEC systems in both tests were deemed reusable after replacing the O-connectors.

Due to the deformation demand on the slab at the rigid wall-floor connection, localized damage occurred to the CIP slab in PFS1. On the other hand, there was little damage occurred to the precast slab in PFS2 except yielding of the strap plates at the plank-beam connections. Overall, damage to the surrounding structure in PFS2 was lighter than that in PFS1.

As shown in Fig. 5 and Fig. 10, the strength of PFS1 was much larger than that of PFS2, which was mainly due to the gravity loads sustained by the wall and the larger resistance from the CIP surrounding structures in PFS1. The energy-dissipation capacity of PFS1 was larger than that of PFS2, but the self-centering performance of PFS2 was better. The damage to the surrounding structures in PFS1 contributed more energy-dissipation, but it also impaired the self-centering of the entire assemblage.

5. Challenges in design practice

It is common sense to researchers and engineering professionals that current design philosophy, which is mostly ductility-based, indicate that development of plasticity in structures (i.e., damage) is inevitable and actually necessary, making it possible for the engineers to design for a reduced lateral force and decrease the construction cost. However, this common sense was generally not valid among other communities, such as residents, building owners, and governors, etc. From their perspective, not only “life-safety” of the structure shall be guaranteed during earthquakes, but also repair cost and business downtime of the structures shall be acceptable after earthquakes. The incompatible expectations on the built structures, which were designed based on the practicing codes, might cause potential risks to the society after earthquake events.

The concept of rocking-wall systems has been proposed and studied extensively in the last three decades. The excellent seismic performance of the systems has been proved by many experimental and numerical studies. However, the system is still not popularized in practice. The main challenges are summarized below.

Currently, there is neither any credit provided by the government for using the rocking-wall system, nor the insurance company would take into account the resiliency provided by the rocking-wall structures. From building owner's perspective, although utilizing this innovative system will create great benefit after seismic events, it might increase the risk of over-budge or schedule-delay due to the uncertainty of a new system.

From general contractor's perspective, constructing structures with rocking-wall systems is generally outside the comfort zone. Because there are few structures of similar types being built so far, it is difficult to gain experience and become familiar with this new system.

From practicing engineer's perspective, although there is some code guidance for the application of the rocking-wall system, it is not as complete as that for regular structure walls. Moreover, most of the design software has not created a specific module to assist the design of this new system.

In summary, in order to popularize the rocking-wall system, or other types of resilient structural systems, it is important to promote the communication and education among all parties, especially the government authorities, for the benefit of using resilient systems in design practice.

6. Conclusion

Two structural assemblages incorporating rocking-wall systems and surrounding structures were successfully tested under quasi-static cyclic loading. PreWEC (Precast Wall with End Columns) was used as the rocking-wall system in both test assemblages. The first assemblage (PFS1) was built using a CIP unbonded PT slab with rigid wall-floor connections to investigate an upper bound wall-floor interaction; the second assemblage (PFS2) was built using a precast slab with special vertical-isolation wall-floor connectors to investigate a lower bound wall-floor interaction. The conclusions derived from the tests are as follows:

(1) Limited and anticipated damage occurred to the PreWEC system in both tests, and the system was deemed reusable after replacing the energy-dissipation elements (O-connectors).
(2) Localized damage occurred to the CIP slab at the rigid wall-floor connection in PFS1; little damage occurred to the precast slab with vertical-isolation wall-floor connectors in PFS2. Either slab was deemed reusable after minor repair.
(3) Both rocking-wall test assemblages exhibit great energy-dissipation and self-centering performance with flag-shaped hysteresis responses recorded in the tests.
(4) Comparison of the two tests demonstrates that the interaction between the PreWEC systems and the surrounding structures has a great impact on the performance of rocking-wall structures, which is mainly determined by the selection of wall-floor connections.

7. Acknowledgements

This research was supported by a grant from National Science Foundation of the United States (CMMI-1041650). The generous support from the following companies is also acknowledged: Molin Concrete provided the precast members; SWPC supplied the PT strands; Hayes Industries provided the wedges for the PT anchorages; BS Italia provided the V-connectors, and JVI provided the Mini V connectors.

8. References

[1] ITG. (2009). Requirements for Design of a Special Unbonded Post-Tensioned Precast Shear Wall Satisfying ACI ITG-5.1 and Commentary. *ACI ITG-5.2-09*, Farmington Hills, MI.

[2] Sritharan, S., Aaleti S., Henry R.S., Liu K.Y. and Tsai K.C. (2015). "Precast concrete wall with end columns (PreWEC) for earthquake resistant design." *Earthquake Engineering and Structural Dynamics*, V. 44, No. 12, pp. 2075–2092.

[3] Aaleti, S., and Sritharan, S. (2011). "Performance Verification of the PreWEC Concept and Development of Seismic Design Guidelines." *ISU-ERI-Ames Report ERI-10347*, Department of Civil, Construction and Environmental Engineering, Iowa State University, Ames, IA

[4] Liu, Q.Z., French C.W., Sritharan S. (2018). "Study of Interaction between a PreWEC System and Surrounding Structure." *ACI Structural Journal*, 115(5), 1407–1418

[5] Liu, Q.Z., French C.W., Sritharan S. (2020). "Performance of A PreWEC System with Precast Surrounding Structure." *ACI Structural Journal* (In press May 2020)

[6] Priestley, M. J. N., Sritharan, S., Conley, J. R., and Pampanin, S. (1999). "Preliminary results and conclusions from the PRESSS five–story precast concrete test building." *PCI J.*, 44(6), 42–67.

[7] Schoettler, M. J., Belleri, A., Zhang, D., Restrepo, J. I., and Fleischman, R. B. (2008). "Preliminary results of the testing for the development of a diaphragm seismic design methodology." *PCI J.*, 54(1),100-124.

[8] Gavridou, S., Wallace, J. W., Nagae, T., Matsumori, T., Tahara, K., and Fukuyama, K. (2017). "Shake-Table Test of a Ful-Scale 4-Story Precast Concrete Building. I: Overview and Experimental Results." *J. Struct. Eng.*, 143(6).

[9] ACI (American Concrete Institute) (2011). "Building code requirement for reinforced concrete." *ACI 318-11*, Farmington Hills, MI.

[10] Victory Connector User Manual. *B.S.Italia, Inc.* 2012

[11] Liu, Q.Z. (2016). "Study on Interaction between Rocking-Wall System and Surrounding Structure." *Ph.D. thesis*, University of Minnesota, Twin Cities, Minnesota, the United States.

[12] ABAQUS user manual 6.11. Dassault Systems, Inc. 2011

