Steel and Composite Shear Walls - Two High-Performance Lateral Force Resisting Systems

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Abstract. This keynote speech focuses on cyclic behavior and seismic design of steel and composite shear walls, two efficient and ductile lateral force resisting systems. First, a summary of types of steel shear walls and their seismic performance are presented. Then, recent advances on the *unstiffened* steel plate shear walls are discussed. Unstiffened steel plate shear walls have been studied, and their design procedures are currently in most seismic design codes. However, their use has been quite limited. The main reason is that in the current unstiffened steel plate shear walls, included in the seismic codes, such as the North American specifications and the Eurocode, quite large lateral forces are applied to the boundary columns creating significant bending moments in the columns. The other reason for reluctance in using the existing steel plate shear wall is the very high cost of the field-welded moment connections that are currently used in this system. The keynote speech will discuss innovative systems developed in recent years to eliminate both problems. The second part of the keynote speech will focus on the steel-concrete composite systems. Available cyclic tests are briefly summarized, and recent developments and innovative systems will be discussed.

Keywords: Steel structures · Seismic design · Steel shear walls · Composite shear walls · Innovative systems

1 Background on Steel Shear Walls

1.1 Introduction to Steel Shear Walls

Steel Shear walls consist of a boundary moment frame and an infill steel plate, which is usually welded, or occasionally bolted to the boundary frame. The infill steel plate can be *stiffened* or *unstiffened*, Fig. 1. The stiffeners can be horizontal, vertical, or both. Diagonal stiffeners have also been studied. The stiffeners usually are steel plates, but, in some cases, steel channels have also been studied, tested, and successfully used. The following sections present information on the behavior and design of *stiffened* and *unstiffened* steel shear walls.

Early applications of steel plate shear walls were *stiffened* steel plate shear walls, such as in the 66-story Nippon Steel Building in Tokyo in the 1960s and the seismic retrofit of military Veterans Hospitals in the United States [1].

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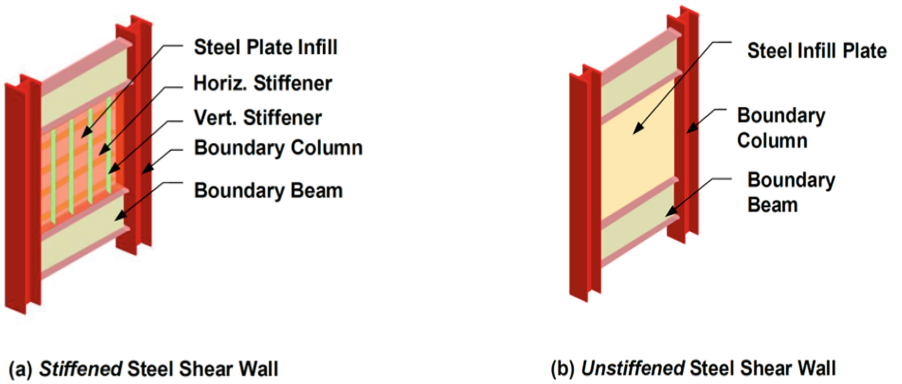

Fig. 1. Typical "Stiffened" and "Unstiffened" steel shear walls.

The tallest steel plate shear wall high-rise is currently the 75-story Jinta Tower in China, with steel shear walls stiffened by using vertical channel section stiffeners [2]. The tallest high-rise with *unstiffened* steel shear walls is the 55-story Los Angeles Convention Center Hotel [3].

The two steel shear wall buildings that have been subjected to major earthquakes and survived with minor damage were the 6-story Sylmar Hospital in Los Angles [1], and the 35-story Kobe City Hall Tower, Kobe, Japan [4].

1.2 Stiffened Steel Plate Shear Walls

The *stiffened* steel plate shear wall system consists of a moment frame with steel plate infills, where the steel plate has vertical, horizontal, diagonal, or horizontal plus vertical stiffeners. The primary role of stiffeners is to provide lateral restraint to the infill steel plate and delay its out-of-plane diagonal buckling. Unstiffened steel infill plates develop diagonal buckling at relatively low story shear forces. Beyond buckling load, the shear wall resists the shear force by Tension Field Action, similar to the behavior of slender webs in plate girders. On the other hand, if sufficient stiffeners are provided, the *stiffened* steel plate shear wall can reach its shear-yielding capacity before its diagonal buckling and the Tension Field Action development. However, suppose the stiffeners are insufficient to fully restrain the infill plate against diagonal buckling. In that case, the *partially stiffened* infill plate will start yielding, but before reaching its total shear yield capacity, it will diagonally buckle and develop diagonal Tension Field Action.

Horizontally and Vertically Stiffened Steel Plate Shear Walls. Zhao and Qiu [5] studied analytically *stiffened* steel plate shear walls with horizontal, vertical, and multi-stiffened (i.e., horizontal-plus-vertical) stiffeners and compared their inelastic pushover behavior to that of *unstiffened* steel plate shear walls. Figure 2 shows the results of this study in terms of shear strength and shear stiffness versus the story drift.

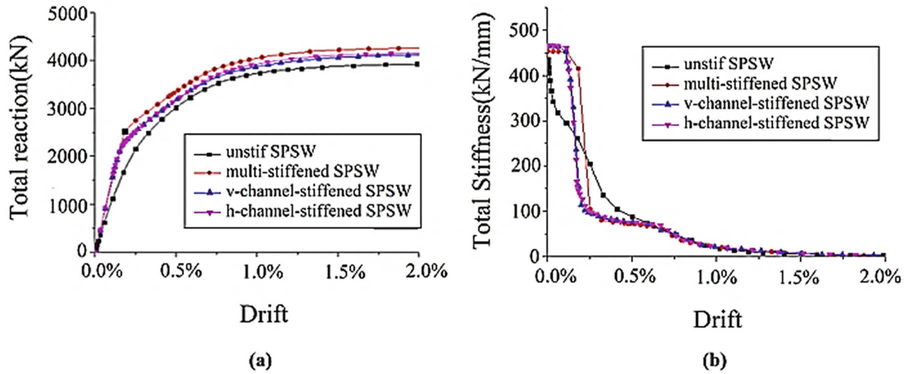

Fig. 2. Comparison of (a) Shear strength versus drift, and (b) Shear stiffness versus drift of three *stiffened* steel shear walls to *unstiffened* steel plate shear wall [5].

As Fig. 2a indicates, the shear strength of the three *stiffened* cases was only slightly more significant than the shear strength of the *unstiffened* case. For stiffness, as Fig. 2b indicates, the initial elastic stiffness of the *unstiffened* case at about 440 kN/mm and the *stiffened* cases at about 450–460 kN/mm was close. But, after shear buckling of the *unstiffened* case at a minimal drift of about 0.02% (0.0002 radians), its shear stiffness drops suddenly and significantly to about 300 kN/mm and continues to decline, albeit at a slower pace, as the tension field action takes over in resisting shear. On the other hand, for the three *stiffened* cases, the initial shear stiffness remains almost unchanged until a drift value of about 0.12% (0.0012 radians), and then suddenly drops to 100 kN/mm.

Some of the findings of the Zhao and Qiu study [5] were:

1. Adding stiffeners to the steel infill plate increased the initial elastic diagonal buckling load significantly (almost 12–13 times), increased ultimate shear capacity only slightly (6%–8%), and reduced the out-of-plane deformation of the paned to almost ½ of the deformation of the *unstiffened* wall;
2. Compared to plate stiffeners, channel section stiffeners were more effective;
3. Considering the relatively high cost of the panel with vertical and horizontal stiffeners, the gain in increasing the strength and stiffness is negligible,
4. When a vertical load is present, the *stiffened* wall with vertical stiffeners showed better performance than the wall with horizontal stiffeners.

Haddad et al. [6] tested *stiffened* and *unstiffened* steel plate shear walls using vertical-horizontal, circular, and diagonal stiffeners. They concluded that adding stiffeners resulted in increasing shear stiffness up to 2.4 times of the unstiffened specimen, increasing initial buckling load, ductility, and energy dissipation capacity, but not increasing shear strength any significant amount.

Diagonally Stiffened Steel Plate Shear Walls. Alavi and Nateghi [7] analyzed and tested two ½-scale, one-story specimens of *unstiffened* and *diagonally stiffened* steel plate shear walls. The *stiffened* shear wall had diagonal stiffeners. Figure 3 shows von Mises stresses, specimen during testing, and shear force drift hysteresis curves for the *unstiffened* and *diagonally stiffened* tested specimens.

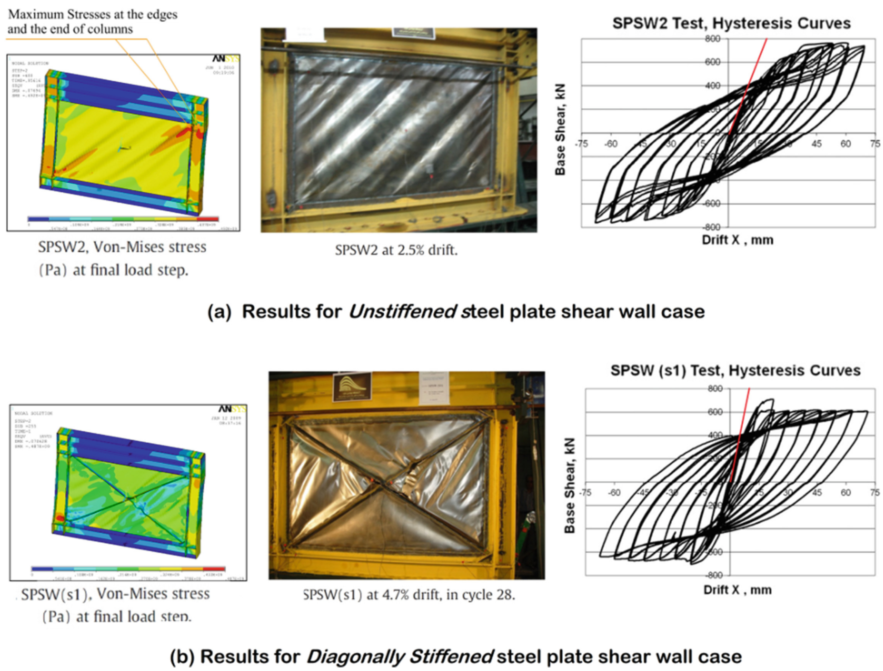

Fig. 3. Comparison of the behavior of (a) unstiffened and (b) diagonally stiffened steel shear walls [7].

The inelastic finite element analysis and test results of the two specimens, as shown in Fig. 3, indicated that both specimens behaved similarly in a ductile manner and could tolerate more than 15 inelastic cycles reaching a maximum inter-story drift of 4.6%. The shear strengths of both specimens were very close. However, the extent of yielding and damage in the *unstiffened* specimen, especially to the infill panel and columns, was more extensive than the damage in the *diagonally stiffened* specimen; see FEA results in Fig. 3. For stiffness, as was the case with the vertical and horizontal stiffeners, in this case also, the presence of the diagonal stiffeners increased the shear stiffness significantly and almost doubled the stiffness of *unstiffened shear wall;* see the red lines on the hysteresis curves in Fig. 3.

Akhavan Sigaroudi et al. [8] also studied and tested diagonally stiffened steel plate shear walls and found similar results. In addition, they propped equations to establish shear strength and stiffness of the diagonally stiffened steel shear walls that predicted the values obtained from their 1/3-scale test specimens with reasonable accuracy.

Trapezoidal Corrugated Steel Shear Walls. Emami et al. [9] tested three ½-scale specimens *unstiffened, vertically-corrugated,* and *horizontally-corrugated* one-story, one-bay steel plate shear walls. Figure 4 shows *unstiffened* and *vertically-corrugated* specimens, before and after the test, as well as their shear force- drift hysteresis curves. The behavior of the horizontally-corrugated specimen was very similar to that of the vertically-corrugated specimen. The tests indicated that the shear yield strength of the corrugated shear walls was about half of the unstiffened shear wall. The maximum cyclic drift of the unstiffened specimen was 4.2%, while the corrugated specimens reached a maximum drift of about 5.8%. The elastic stiffness of the corrugated specimens was about 25% higher than that of the unstiffened specimens.

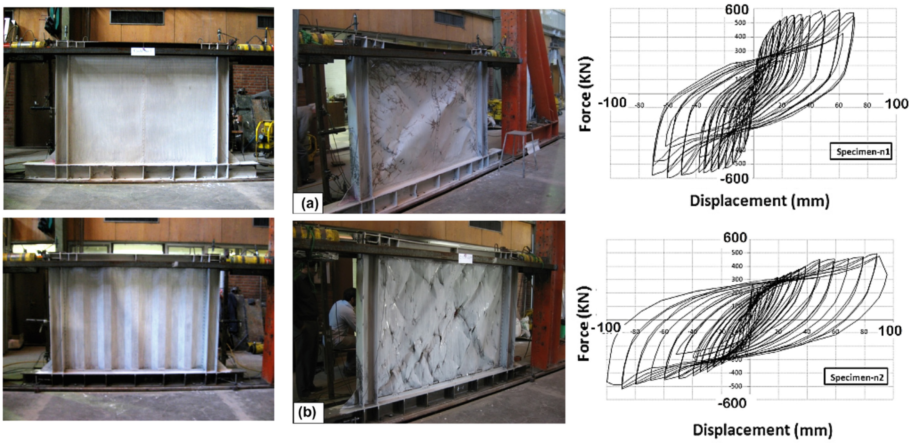

Fig. 4. Comparison of unstiffened steel shear wall behavior (top row) to the vertically-corrugated steel plate shear wall behavior (bottom row) [9].

1.3 Unstiffened Steel Shear Walls

A typical *unstiffened* steel plate shear wall system consists of a boundary moment frame with steel infill plates welded to the boundary columns and beams.

The Behaviour of Unstiffened Steel Plate Shear Walls. Several researchers, among them [10–13], and [14], have studied the behavior of unstiffened steel shear walls under monotonic and cyclic lateral force applications experimentally. Figure 5 shows, schematically, the shear force-drift response of a typical *unstiffened* steel plate shear wall. The shear force-drift curve is linear elastic from Point O to Point A. The unstiffened infill plate diagonally buckles at Point A, usually at relatively low shear force and drift levels. After buckling, the shear force in the steel infill plate is primarily resisted by the plate's Tension Field Action (TFA). Due to buckling at Point A, the shear stiffness of the system decreases significantly [15]. Buckling of an unstiffened shear wall does not reduce its shear yield capacity much, and the TFA can develop almost the same shear yield strength. However, due to buckling and reduction of shear stiffness, the shear yield capacity is reached in a larger drift value. As a result, the shear force-drift hysteresis curves in unstiffened steel plate shear walls have pronounced pinching and reduced energy dissipation capacity.

From Point A to Point B in Fig. 5, the shear force drift curve is again almost elastic, albeit with lower shear stiffness and local yielding within the steel plates and the boundary frame. At Point B, diagonal Tension Field areas of the plate have yielded significantly, and we can observe a noticeable decrease in the shear stiffness. Beyond Point B, the steel plate experiences strain-hardening, and the boundary frame continues to develop more yielding.

At Point C in Fig. 5, the Tension Field develops fracture, shear capacity starts to drop, and the wall fails.

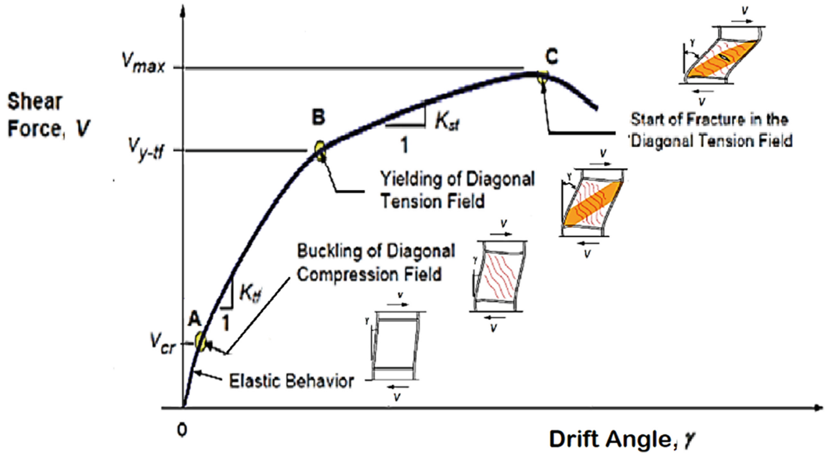

Fig. 5. Shear force-drift behavior of typical *unstiffened* steel plate shear walls.

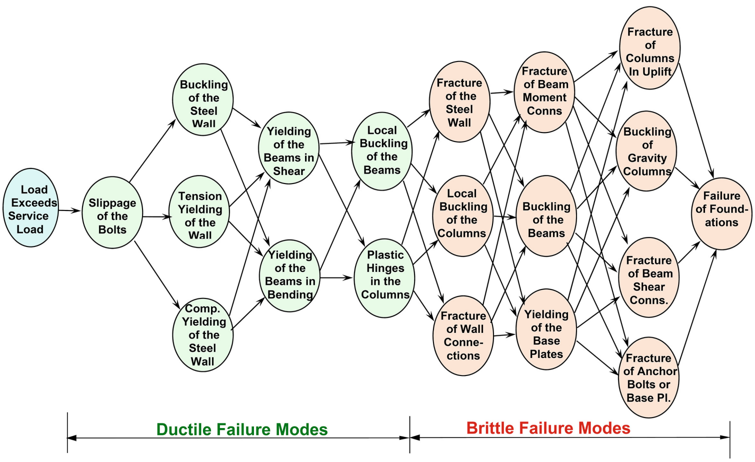

Fig. 6. Hierarchy of failure modes of steel plate shear walls regarding *ductile* and *brittle* failure modes [1].

The available research and actual cyclic testing results indicate that adequately designed steel plate shear walls are quite ductile and can reach cyclic drift of more than 4% [16]. However, to exhibit such behavior, the governing failure mode should be a *ductile* failure mode such as the yielding of the steel plate and not a brittle one such as the fracture of the wall. Figure 6 above shows failure modes of a typical *unstiffened* steel shear wall [1] placed in a "hierarchical" manner in terms of the time of their occurrence. The *ductile* failure modes are on the left. The design philosophy and procedures for steel shear walls must ensure that this hierarchy is maintained and that none of the *brittle* failure modes occur before the ductile failure modes.

1.4 Problems of Unstiffened Steel Shear Walls

Based on available research, the steel plate shear wall that is designed following the current seismic codes such as the AISC Seismic Provisions [17] will have high ductility, energy dissipation capacity, and sufficient lateral stiffness and shear strength. However, its use in actual buildings has been limited. Several problems with the steel plate shear walls (SPSW) included in the AISC Seismic Provisions [17] make the system quite expensive compared to other steel or reinforced concrete lateral force resisting systems. Figure 7 shows some of the main problems.

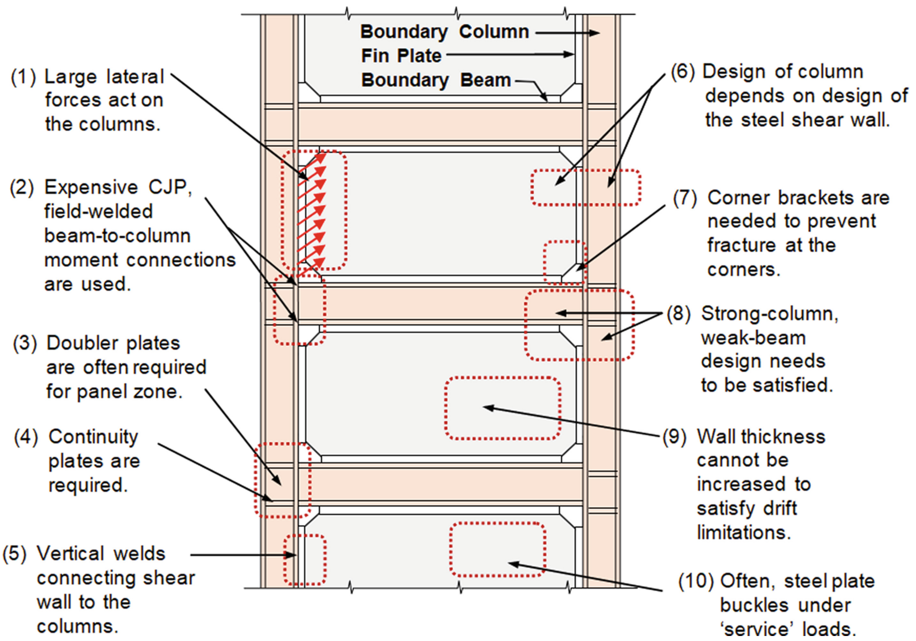

Fig. 7. Some of the issues that make the *unstiffened* steel plate shear wall system in the seismic design codes uneconomical compared to other lateral force resisting systems [18, 19].

In the next section, we discuss a few innovative steel plate shear walls developed and some used in actual buildings to address some or most of the problems shown in Fig. 7 above.

1.5 Innovative Solutions to Remove Deficiencies of the Current Unstiffened Steel Walls

Use of Low Yield Steel as Infill Plate. Nakashima et al. [20] and Nakagawa et al. [21] conducted some of the early studies of the use of Low Yield Steel (LYS) in steel plate shear walls. The specimens were *stiffened* and had vertical stiffeners on one side and horizontal stiffeners on the other side of the panel. The use of the Low Yield Steel infill panels resulted in higher ductility and energy dissipation capacity and earlier yielding. Tests by Vian et al. [22] of steel shear walls with Low Yield Steel infill panels also confirmed these behavior traits. The SPSW with a Low Yield Steel infill panel is a viable alternative to the traditional unstiffened SPSWs and can alleviate the over-strength problem on the adjacent framing members.

The "Skilling" Innovative System. Figure 8a shows the innovative steel shear wall system developed and used by Skilling Ward Magnusson Barkshire (now Magnusson Klemencic Associates). In this system, two *unstiffened*, one-bay steel shear wall systems are coupled by steel beams on each floor. All connections are "Special" moment connections.

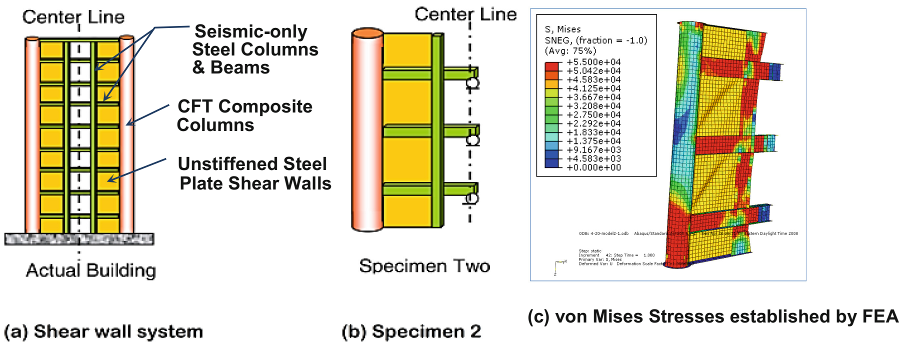

Fig. 8. (a) The innovative system was developed by MKA and tested by Zhao and Astaneh-Asl, [16], (b) Specimen 2, and (c) Von Mises stresses for Specimen 2.

As shown in Fig. 8a, the steel columns and beams in this system are "seismic-only" elements, which means they have very small, if any, gravity load in them. The gravity load applied to the system is resisted primarily by relatively large CFT composite columns, see Fig. 8a. As a result, the steel columns in this system are designed to undergo severe yielding during the design earthquake and be an energy-dissipating element along with the steel beams and the steel plate shear wall itself. The main innovation in this system is that, unlike the current *unstiffened* shear wall system in the AISC Seismic Provisions [17], where the columns should remain elastic, in this innovative system, the “seismic-only” steel columns, which do not have gravity loads, are allowed to yield and bend inelastically and dissipate energy.

Two ½-scale specimens of this system were tested at the University of California, Berkeley by Zhao and Astaneh-Asl [16]. One specimen was one-story, with ½-stories above and below, and the second specimen had 2-stories with two ½-stories above and below. Figure 9 shows the results for the 2-story Specimen 2. The specimen is shown in Fig. 8b earlier.

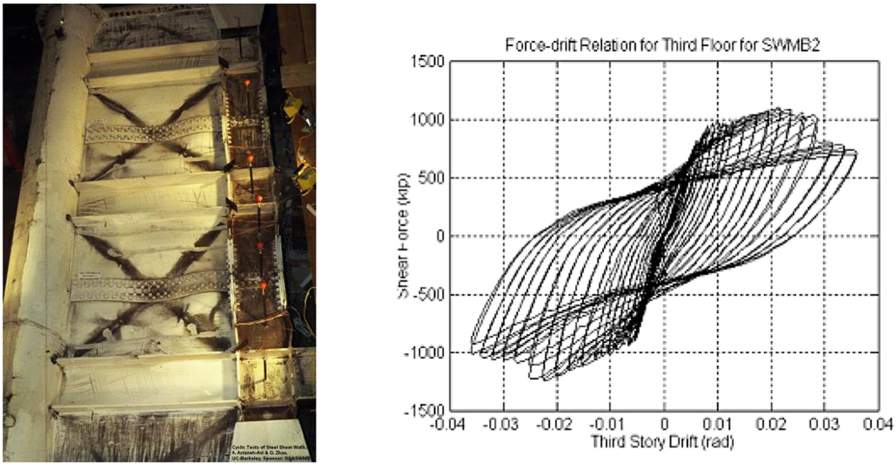

Fig. 9. Specimen 2 at the end of the test (left) and its second-floor cyclic shear force versus drift behavior [16].

Both specimens behaved in a very ductile manner. Figure 9 (left) shows Specimen 2 at the end of the test. The dark color of the “seismic-only” columns on the right side indicates significant yielding of the steel “seismic-only” columns. The significant yielding was primarily due to the application of Tension Field Action forces on these columns after diagonal buckling of the steel plate shear walls. Figure 9 on the right shows the cyclic shear force-drift response of the second floor of the 2-story Specimen 2. At about 0.55% and 0.6%, the shear walls in the one-story and two-story specimens buckled, respectively. At 2.8% and 2.2% drift, the maximum shear strength of Specimens 1 and 2 reached, and the shear strength dropped. In both specimens, the drop in shear strength was due to fracture of a coupling beam just outside its connection to the “seismic only” steel columns, Fig. 9.

Steel Shear Walls with Slits. In this *unstiffened* shear wall system, the innovation is to cut vertical slits into the infill plate. The slits turn the infill plate into several shear-flexural narrow vertical plates called *links*. As a result, the out-of-plane diagonal buckling and development of tension field action are delayed or prevented altogether before the shear yielding of the steel infill plate. Elimination of the tension field action results in not having lateral tension field action forces acting on the boundary columns, which is a significant problem in the current *unstiffened* steel plate shear wall system.

Hitaka and Matsui [23] presented the results of 42 tests of steel shear walls with slits 1/3-scale specimens subjected to static monotonic and cyclic lateral loading. They concluded that before the unset of the out-of-plane buckling of the plate with the slits, the yield mechanism was primarily due to the flexural and shear yielding of the plate links. As a result, the hysteresis curves show no pinching effect, Fig. 10a. They also concluded that if the width to thickness ratio of the links is less than 20, the wall can sustain roughly 3% drift without hysteresis degradation Fig. 10a. However, the hysteresis curves were pinched for specimens whose out-of-plane diagonal buckling occurred before the in-plane flexural/shear yielding, as shown in Fig. 10b.

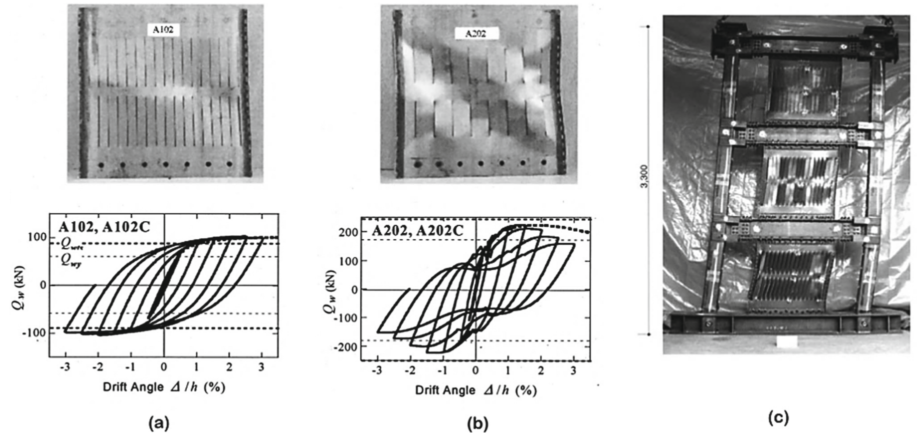

Fig. 10. (a) Specimen with significant in-plane flexural/shear yielding, (b) specimen with out-of-plane diagonal buckling causing pinching of the hysteresis curves [23], and (c) a three-story frame specimen after the test [24].

Hitaka and Matsui [23] concluded that: (a) the behavior is ductile and stable; (b) strength and stiffness can be adjusted independently, and (c) the wall need not be connected to the boundary columns.

Cortes and Liu [25] conducted ten cyclic tests of 1/3-scale steel shear walls with slits. All specimens were capable of undergoing inter-story drifts of at least 5% without reducing the load-carrying capacity below 80% of ultimate strength, which was considered the state of failure by the authors.

The New High-Performance Steel Plate (HPSPSW) Shear Wall System. Figure 11 shows the main components of this innovative system developed and proposed by Qian and Astaneh-Asl [18] and [19]. The system addresses the three most important problems of the *unstiffened* steel plate shear walls currently in AISC Seismic Provisions [17], which are:

1) large columns resulting from the shear wall applying lateral Tension Field Action loads to the columns,
2) expensive field-welded moment connections, and
3) buckling of the steel plate under service earthquakes and winds.

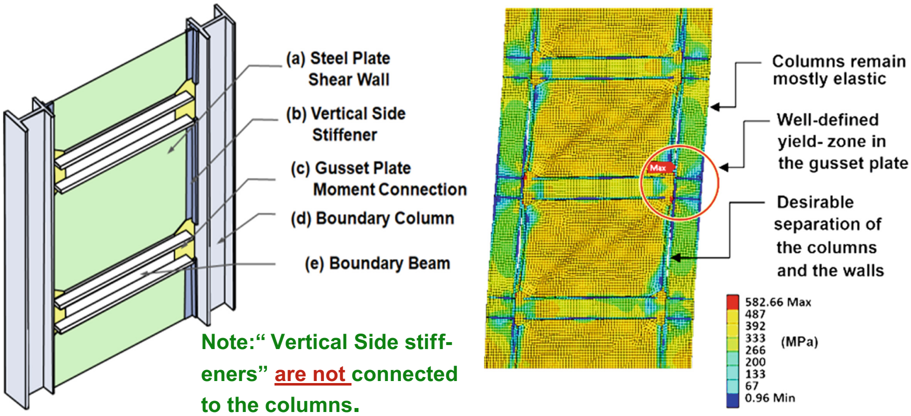

Fig. 11. Main components of the new High-Performance Steel Plate Shear Wall system (left) and von Mises stresses at a drift ratio of 2.22 [18, 19].

Figure 11 on the right side shows the von Mises effective stresses for an HPSPSW model at a roof drift ratio of 2.22%. As the figure shows, the columns remain predominantly elastic since, in this system, there are almost no lateral Tension Field Action forces applied to the boundary columns. The reason is that the steel plate shear wall is not connected to the columns and only is connected to the beams. Figure 11 also shows that the new gusset plate moment connection (GPMC) can develop a clear plastic hinge and yield zone. This yield zone, as designed, acts as a ductile fuse in the new GPMC and protects all other elements of the joint, including beams, columns, weld lines, and bolts from yielding and fracture. As a result of the formation of a plastic hinge in the Gusset Plate Moment Connection, in this system, there is no need to satisfy the "Strong Column-Weak Beam" requirement in the moment frames. The "Strong Column-Weak Beam" requirement, in many cases, causes the columns in a moment frame to be stronger than needed, adding to the cost of the construction.

The main feature of the new HPSPSW system is that the two vertical edges of the steel plate shear wall are not connected to the boundary columns; instead, the vertical edges are connected to stiffeners placed next to the columns. As a result of not connecting the steel plate shear wall to the columns, no Tension Field Action forces are applied to the columns. In addition, separating the steel plate shear walls from the columns allows the columns to be any steel or composite section, even any reinforced concrete cross-section, as shown in Fig. 12. Note that in the current steel plate shear wall system included in the AISC Seismic Provisions [17], the boundary columns are steel wide flanges, and the steel plate is welded to the flanges.

Figure 12 also shows a variety of "edge stiffeners" that can be used at the two vertical edges of the steel plate in HPSPSW. For more information on the performance of various edge-stiffeners, see Qian and Astaneh-Asl [19].

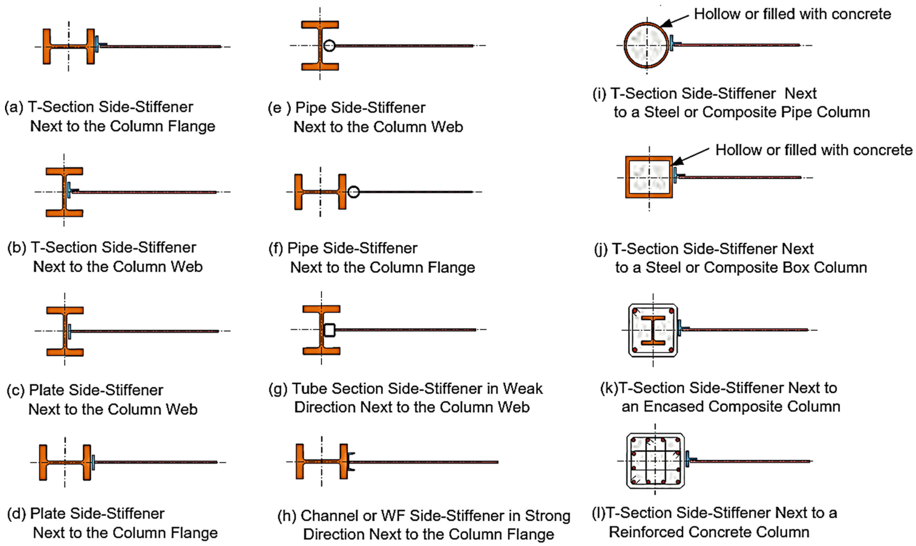

Fig. 12. Examples of a main components of the innovative High-Performance Steel Plate Shear Wall developed by Qian and Astaneh-Asl [19]

In this system:

1) The steel plate shear walls are not connected to the boundary columns, which results in almost no lateral tension field action forces applied to the columns
2) The beam-to-column connections in this system are themselves an innovative and cost-effective connection also developed by Qian and Astaneh-Asl [19] and [26], called Gusset Plate Moment Connection; and,
3) The thickness of the infill plate is independent of the column size, enabling the use of thicker plates if needed to prevent buckling of the steel plate under service loads. In the current steel plate shear wall system, any change in the thickness of the wall plate results in a proportional increase in the lateral tension field action forces applied to the column, which makes the columns even heavier and more expensive.

Figure 13 shows ten improvements that the innovative HPSPSW system has over the current steel plate shear wall in the current seismic design codes such as the AISC Seismic Provisions [17].

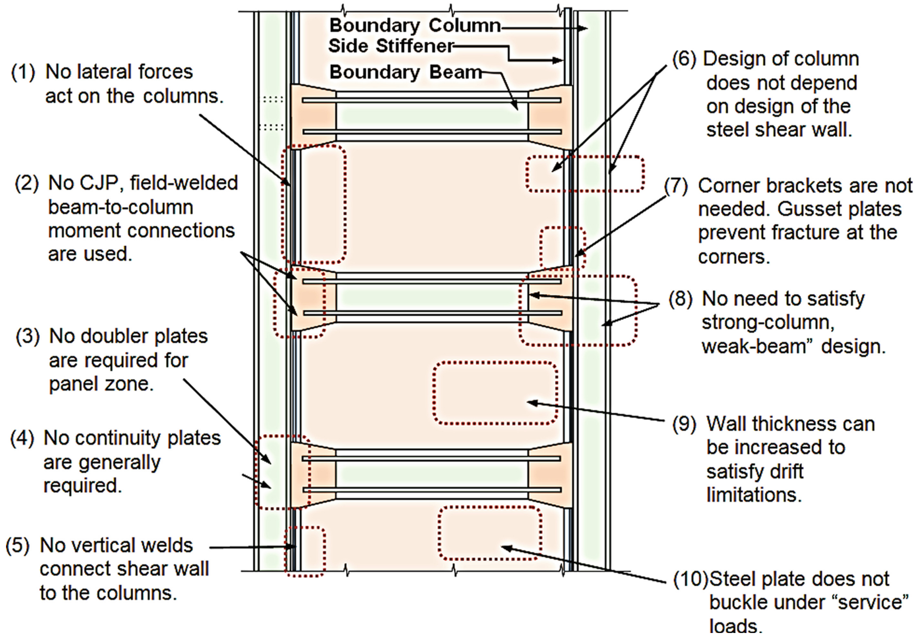

Fig. 13. The ten advantages of the innovative High-Performance Steel Plate Shear Walls (HPSPSW) developed by Qian and Astaneh Asl [18, 27].

Figure 14 shows welded and bolted versions of the innovative Gusset Plate Moment Connection used in the High-Performance Steel Plate Shear Wall system. More information on the HPSPSW and the GPMC is in Qian and Astaneh-Asl [19] and [26]. The innovative Gusset Plate Moment Connection is not just for use in the boundary frames of the steel shear walls, but it can be used in the special moment frames, dual concentrically braced frames, and eccentrically braced frames.

Ghamari and Haeri [28] studied the inelastic pushover behavior of current SPSW and HPSPSW (with A36 steel plate and Low Yield Steel plate). They concluded that the HPSPSW with Low Yield Steel Plate exhibits higher properties than SPSW and HPSPSW with the A36 steel plate in both elastic and inelastic zones.

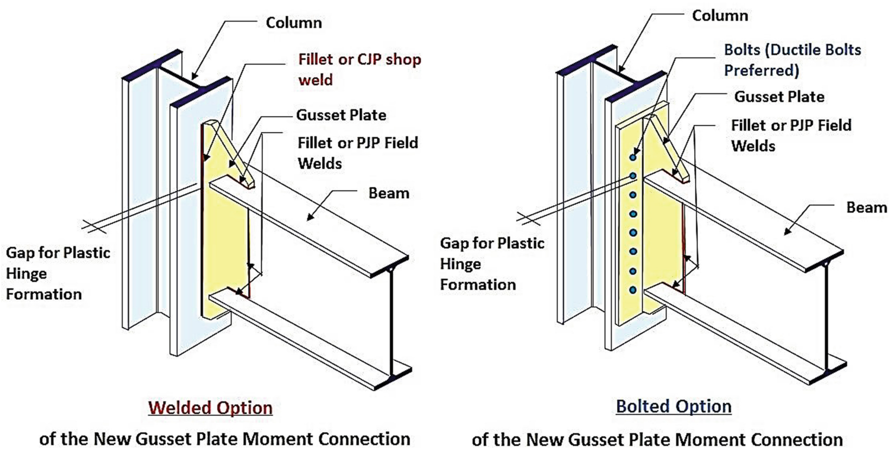

Fig. 14. Welded Option (left) and Bolted Option of the innovative Gusset Plate Moment Connection developed by Qian and Astaneh-Asl [26].

Figure 15 shows cyclic moment-rotation behavior of welded Gusset Plate Moment Connection and von Mises stresses in the boundary beam at maximum moment capacity point. The beam is essentially elastic, with ductile plastic hinges forming in the new Gusset Plate protecting all other elements of the system from yielding.

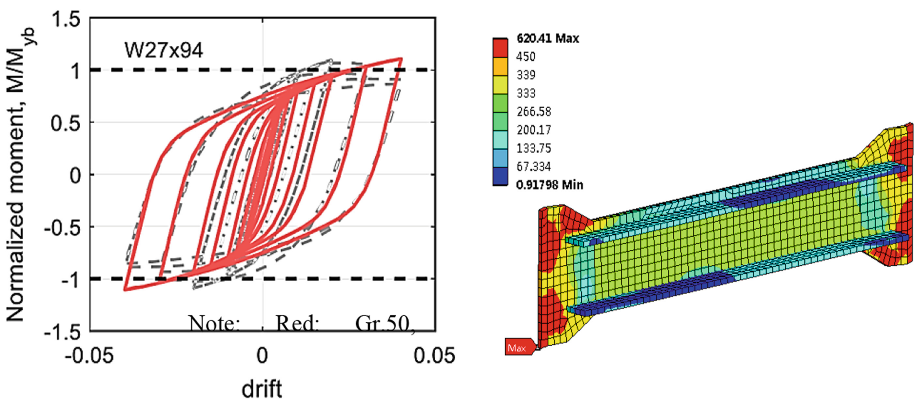

Fig. 15. Typical cyclic moment-rotation curves (left) and von Mises stresses (in MPa) for welded Gusset Plate Moment Connection at maximum moment point [26, 27].

1.6 Composite (Steel-Concrete) Shear Walls

Typical composite shear walls consist of two steel plates sandwiching a reinforced concrete wall or a single or two reinforced concrete walls attached to one or both sides of a steel plate, Fig. 16. In the composite shear wall system, currently in the AISC Seismic Provisions [17], the role of the reinforced concrete wall(s) is to provide lateral restrainer to the *unstiffened* steel shear wall and prevent its diagonal buckling until the steel shear wall yields in shear. The reinforced concrete wall(s) are not designed to carry the gravity load. In these cases, the shear capacity of the composite shear wall is equal to the shear yield capacity of the steel plate(s), ignoring the shear resistance of the reinforced concrete wall(s) [17]. However, in recent years, especially in China, composite shear walls are used not only to carry shear, but the reinforced concrete part of the composite shear walls also carries a significant amount of the gravity load.

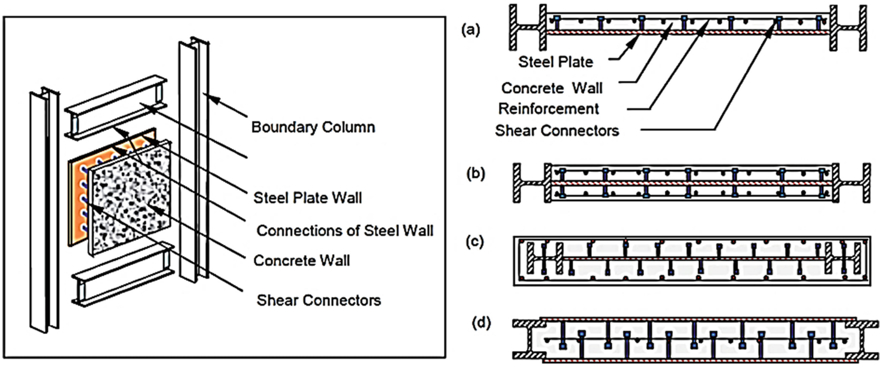

Fig. 16. Components of a composite shear wall (left) and typical cross-sections [29]

The Behaviour of Composite Shear Walls. Actual test results on composite shear walls are minimal. Zhao and Astaneh-Asl [30] conducted cyclic tests of two ½-scale composite shear walls with a cross-section shown in Fig. 15a. Figure 17 shows a composite shear wall specimen at the end of the test and shear force-drift hysteresis curves. The specimen behaved almost elastically until 0.036% drift when the concrete walls in both floors developed major cracks and crushed at the corners. At 2% drift, visible racks could be seen on the reinforced concrete wall. As cyclic loading continued, the reinforced concrete wall continued to sustain damage but was able to prevent diagonal buckling of the wall until a drift of 4%, when the diagonal buckling of the steel plate shear wall started, and the shear strength started to drop.

Zhao and Astaneh-Asl [30] also tested an innovative specimen to improve behavior and reduce the damage to reinforce concrete stiffening wall(s). The specimen was precisely similar to the specimen in Fig. 17 but had a gap of 32 mm between the reinforced concrete wall edges and the steel boundary frame. With a gap around the concrete panel, this specimen behaved in a very ductile and desirable manner. The specimen tolerated 33 cycles, of which 27 cycles were inelastic. The drift at the maximum shear load was 4.4%. When the load dropped to 80% of maximum shear, which is considered the “failure” point, the drift was about 5% (Fig. 18).

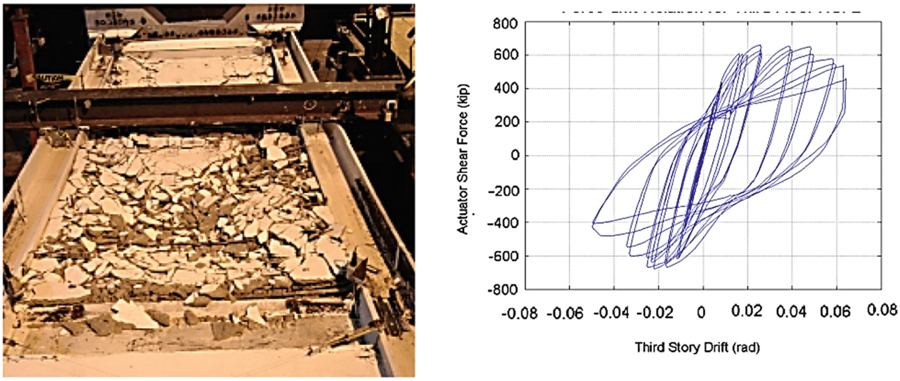

Fig. 17. The composite shear wall at the end of the test and its shear force-drift response [30].

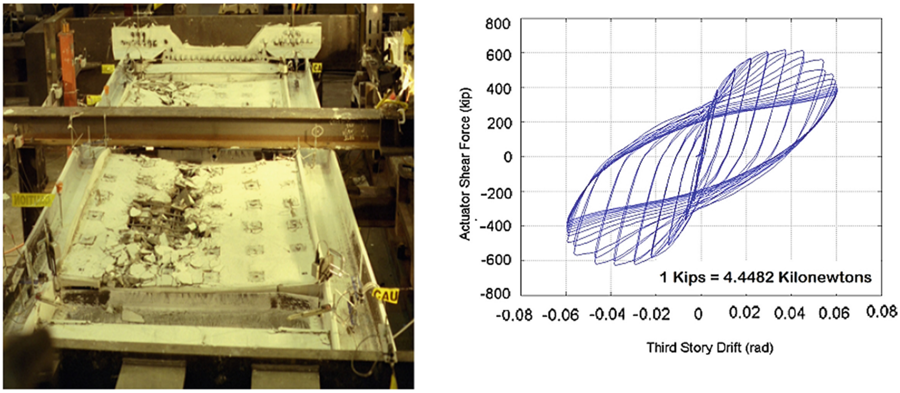

Fig. 18. The composite shear wall specimen with a gap between the steel frame and R/C wall at the end of the test and its shear force-drift response [30].

2 Summary and Conclusions

1. Steel shear walls are a ductile, lateral force resting system with sufficient strength and stiffness to be used economically to resist seismic and other lateral forces.
2. The *stiffened* steel shear walls can be designed not to buckle diagonally before reaching the shear wall's shear yield strength.
3. Diagonally *stiffened* steel plate shear walls show good ductility and energy-dissipation capacity and are an efficient lateral force-resisting system.
4. The use of Low Yield Steel can considerably improve the behavior of steel plate shear walls.
5. The *unstiffened* steel plate shear walls are ductile, energy dissipating systems that resist lateral forces by diagonal buckling under applied shear and developing Tension Field Action.

6. Development of the Tension Field Action in *unstiffened* steel plate shear walls results in relatively large lateral and vertical loads applied to the columns, making the columns quite heavy compared to the columns of the *stiffened* steel shear walls. If *stiffened* steel shear walls are designed not to buckle before yielding, almost no lateral loads are applied to the columns by the shear wall.
7. Other than the Tension Field Action causing heavy columns in *unstiffened* steel shear walls, another reason for the relatively high cost of the current steel plate shear wall in the AISC Seismic Provisions [17] is the use of field-welded moment connections requiring Complete Joint Penetration (CJP) welds.
8. Steel plate shear wall with slits is an innovative and efficient solution to prevent diagonal bucking and the development of Tension Field Action in *unstiffened* steel shear walls.
9. Another innovative and efficient *unstiffened* steel shear wall system is the High-Performance Steel Plate Shear Wall (HPSPSW) system developed by Qian and Astaneh-Asl [19]. In this system, the vertical edges of the steel plate shear wall are <u>not</u> attached to the columns, thus eliminating any Tension Field Action forces applied to the columns. Another innovation in this system is developing and using innovative Gusset Plate Moment Connections (GPMC) instead of relatively expensive CJP field-welded moment connections.
10. Composite shear walls are ductile and efficient lateral force resisting systems capable of exceeding inter-story drift of 4% without any reduction in their shear strength. Furthermore, by providing a gap of about 32 mm between the reinforced concrete wall and the steel boundary frame, the composite wall's behavior was improved, especially since the damage to the concrete wall was much less than the specimen without the gap.

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