Application of Steel Shear Walls Toward More Resilient Structures

Abolhassan Astaneh-Asl, Xin Qian and Yongjiu Shi

1 Introduction

Steel shear wall systems are one of the main steel lateral force-resisting systems in buildings. Other commonly used steel lateral force resisting systems are the moment frame, concentrically braced frame, and eccentrically braced frame. Figure 1 shows the main components of a steel plate shear wall, which are the steel infill plate, the surrounding boundary beams and boundary columns, connections of the infill plate to boundary columns and beams, beam-to-column connections, stiffeners if any, the base connections of the columns, and the splices of the boundary columns.

Steel shear walls are divided into *stiffened* and *unstiffened*. In unstiffened shear walls, the infill plate has no stiffeners, as shown in Fig. 2a, while in the stiffened steel shear walls, there are horizontal or vertical stiffeners on one side, or both horizontal and vertical stiffeners either on one side or each on one side of the steel infill plate, Fig. 2b.

Early steel shear walls used in the 1960s and ’70s were stiffened to prevent buckling of relatively thin infill plates until the plate yields in shear. Later, during the 1980s, the thin unstiffened steel plate shear walls became popular after the post-buckling capacity from the diagonal tension field action of the unstiffened infill plates was recognized. The superior shear resistance, stable hysteresis behavior, high-energy dissipation capability, and high ductility, as well as the inherent redundancy, have made the steel plate shear walls a promising alternative to conventional lateral load-resisting systems in high seismic and wind regions. Unstiffened shear

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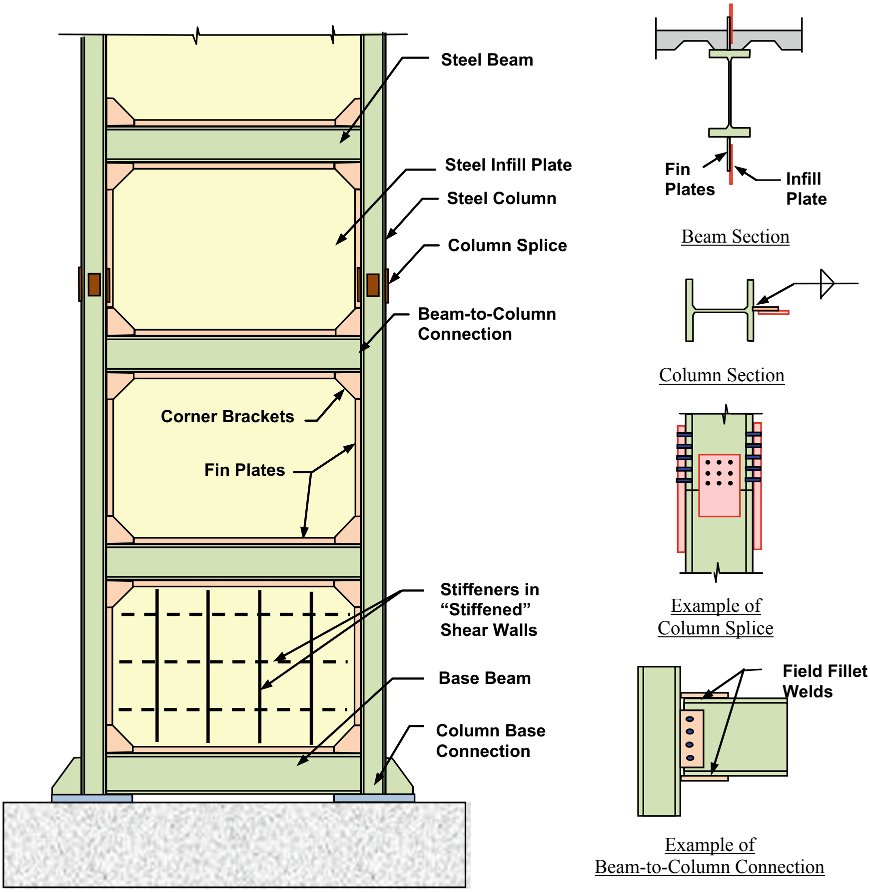
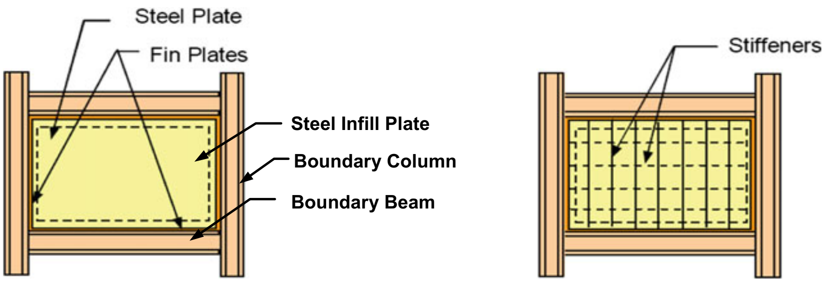


Fig. 1 Main components of a typical steel shear wall system



(a) *Unstiffened* Steel Shear Wall (b) *Stiffened* Steel Shear Wall

Fig. 2 Components of typical unstiffened and stiffened steel shear walls

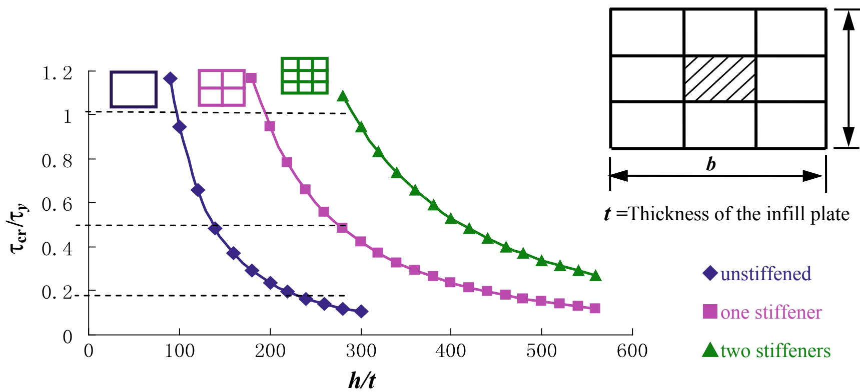


Fig. 3 Shear buckling strength of unstiffened and stiffened steel shear walls

walls are quite popular in the U.S. and Canada, while the steel shear walls used in China and Japan are often stiffened.

The stiffened steel shear walls, compared to unstiffened steel shear walls are more expensive, due to the added cost of stiffeners, and occupy more floor space, again, because of the presence of stiffeners. The loss of usable space can result in a substantial financial loss, especially in high rise buildings.

Figure 3 shows the increase in shear strength of the unstiffened steel plate shear wall when stiffeners added (Shi and Astaneh-Asl 2008). The study indicated that for unstiffened shear walls when height-to-thickness ratio, *h*/*t*, is more than 300, the critical buckling stress of the wall is less than 10% of its shear yield stress. In most applications, today, the height/thickness ratio of unstiffened shear walls is much more than 300 and in the order of 700–1000. The current U.S. code for unstiffened steel shear walls (AISC 2016a) ignores the buckling capacity of the unstiffened shear walls, and only considers the diagonal tension field capacity in the design. As Fig. 3 (Shi and Astaneh-Asl 2008) shows, for *h*/*t* of 300 adding one or two stiffeners in vertical and horizontal directions, increased the shear buckling stress to 45 and 95% of the shear yield stress respectively. The curves indicate that to achieve a critical shear buckling stress equal to yield stress, which will result in yielding of the panel before its buckling, the height to thickness ratio of the unstiffened wall needs to be 100 or less. For stiffened walls, to ensure yielding of the panels before buckling, the same limitation applies but this time to the height-to-thickness of the panel bounded by the stiffeners.

2 Steel Shear Wall Systems

Steel shear walls are usually placed around the elevator and staircase core of the building, although in some cases they have been placed in the other bays even on the façade. Figure 4 shows common steel shear wall systems, which are single bay,

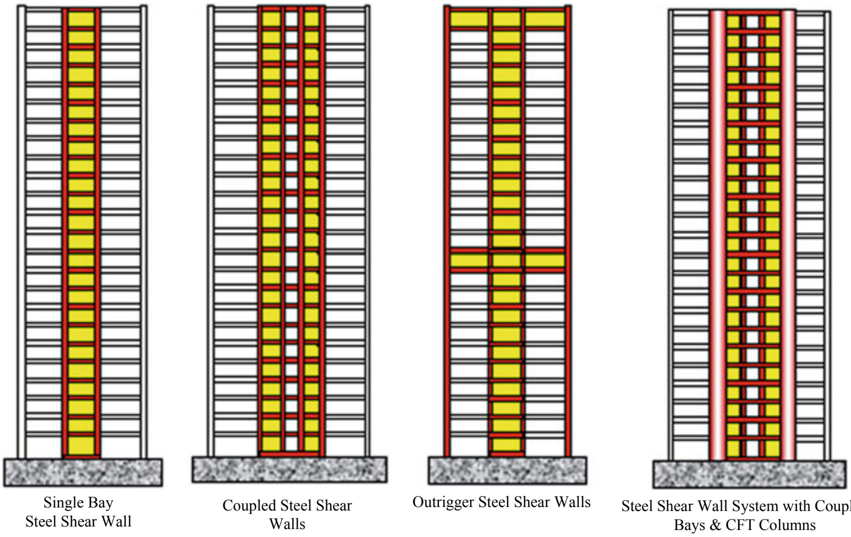


Fig. 4 Typical steel shear wall systems

coupled, outrigger and coupled bays and concrete-filled tubes, respectively. The shear wall in these systems can be either stiffened or unstiffened. The columns and the beams are usually I-section wide flanges with the web of the section in the plane of the steel plate, and the wall connected to the flanges of the columns and beams. The reason for the use of wide flange beams and columns is because of almost all early tests of steel shear walls in North America are done using such wide flange sections. However, in Japan, concrete-filled or hollow structural steel round columns are used as the boundary columns of steel shear wall systems.

The beam-to-column connections in a steel shear wall system are usually a rigid connection required by the current seismic codes. More information on beam-to-column connections in steel shear wall systems is given later in this chapter.

The most common steel shear wall system is the single bay system, where steel plate shear wall is field-welded or bolted to boundary columns and beams. In this system, the steel plate shear wall is designed to resist the entire story shear and the boundary moment frame is designed to resist overturning moment. If the steel shear wall is unstiffened, it resists the story shear by developing a diagonal tension field. The tension field action applies lateral forces to the boundary columns and beams causing shear forces and bending moments in them. The lateral forces applied to the beams from the story below and above are usually slightly different and almost cancel each other, except the roof and base beams that are subjected to lateral load only from below or above respectively. If the steel shear wall is stiffened such that the steel plate within the stiffened panels will yield before buckling, there will be no tension field action and no lateral load on the boundary beams and columns.

In a *single bay steel shear wall*, the steel plate is connected to the vertical boundary columns and horizontal boundary beams. The beam-to-column connections are generally moment connections as required by the seismic design codes such as the AISC-341 standard (AISC 2016a). The moment connections do not need to be a "Special" moment connections capable of accommodating story drift angle of at least 0.04 radians and can be "Intermediate" moment connections. Seismic design codes such as the AISC-341 standard (AISC 2016a) define the Special and Intermediate moment connections. The shear wall is designed to resist the story shear. The entire moment frame resists overturning moment as well as the gravity loads.

In a *coupled steel shear wall system*, two or more single bay steel shear walls are connected with coupling beams. The connections of coupling beams are rigid ductile moment connection. In high seismic areas, the connections should be "Special" moment connections as defined in seismic codes such as the AISC-341 standard (AISC 2016a) and are developed and expected to undergo cyclic plastic rotations of at least 0.03 radians. In coupled shear wall system, the single bay shear walls together with the coupling beams are designed to resist the story shear, and the tension and compression in the single bay shear walls resist overturning moment.

In an *outrigger steel shear wall system*, in a few floors, horizontal shear walls, or trusses, connect the shear wall to the exterior columns. The connections of the outriggers to the columns are usually rigid, but if the length of the outrigger is relatively short, the connections of the outrigger to the exterior columns can be simple (i.e., shear) connections to save in the cost of design, construction, and inspection associated with moment connections. In high seismic areas, the beam-to-column connections should be "Special" moment connections as per governing seismic code such as the AISC-341 standard (AISC 2016a). In an outrigger steel shear wall, the single bay shear walls are designed to resist the story shear only. The entire moment frame including the axial strength of the exterior columns, brought into action by the outriggers, resists the overturning moment as well as the gravity loads.

A *Coupled Bays and Concrete-Filled Tube (CFT) Columns* is an efficient steel shear wall system, originally developed and used by MKA structural engineering firm (Seilie and Hooper 2005) Zhao and Astaneh-Asl (2004, 2008) tested the system under cyclic loading and established its behavior. The system is different from the three systems mentioned above. The two large Concrete Filled Tube (CFT) columns on the right and left side of the system carry a relatively large seismic shear and vertical gravity load. The columns, beams and steel plate shear walls between the two CFT columns primarily carry the seismic load and a negligible amount of gravity load. Unlike all systems mentioned above, where the rule of "strong-column, weak beam requires that the columns remain elastic, in this system the steel columns between the two CFT composite columns are allowed to undergo significant yielding and plastification. More information on this system is provided later in this chapter.

3 Advantages and Disadvantages of Steel Plate Shear Walls

3.1 Advantages of Steel Plate Shear Walls

Compared to other lateral force resisting systems for buildings, such as steel braced or moment frames, or reinforced concrete shear wall or moment frame systems, the main advantages of steel shear wall systems are:

1. Steel shear wall systems have a relatively high shear strength-to-weight ratio compared to reinforced concrete shear walls, which results in significant reduction of the self-weight of the structure reducing gravity and seismic forces in the structure and its foundations.
2. The high ductility and energy dissipation capacity of steel shear walls makes them one of the most efficient lateral force resisting systems in high seismic areas.
3. Since steel shear walls, especially the unstiffened shear walls, have a very small footprint, the thickness of the finished wall is relatively small compared to other systems, enabling more plan areas used as the occupiable floor areas. The increase in useful floor areas can be quite significant in high-rise buildings.
4. Through prefabrication of the steel shear wall units in the shop and having bolted field splices, the costly and time-consuming field weldings can be avoided. Also, eliminating field welding enables the system to be constructed efficiently in cold weather.
5. The infill plates in a steel shear wall system are the elements that experience buckling and yielding during a major seismic event with some yielding expected in the boundary beams as well. The damaged panels can be easily replaced with new panels, and seismic resistance of the steel shear wall system can be restored relatively rapidly and economically.
6. Steel shear walls can be used in new steel and composite structures and even in new reinforced concrete buildings as the main lateral force resisting system. Also, as the past applications indicate, steel shear walls can be used efficiently in the retrofit of existing seismically deficient structures such as non-ductile reinforced concrete and masonry buildings. Examples are Veterans Administration Medical Center and Oregon State Library (Baldelli 1983; Robinson and Ames 2000).
7. As for stiffness, the shear stiffness of the stiffened shear walls, where buckling of the plate is prevented until shear yielding of the plate occurs, can be quite high. However, for unstiffened steel shear walls, due to diagonal buckling of the unstiffened plate under relatively small story drift values of about 0.005, the stiffness can be reduced considerably. The unstiffened steel plate shear wall is stiffer than a steel moment frames but is more flexible than a typical steel braced frame.

3.2 Issues in Using Steel Shear Walls

The following items are important issues in selecting steel shear walls as the lateral force resisting system.

1. A typical steel shear wall covers the entire width and height of the bay. From the architectural point of view, this can be considered as a disadvantage, especially if compared to moment frames where the bay is not obstructed. For this reason, in many applications, steel shear walls are placed around the elevator/staircase core of the buildings. If necessary, steel shear walls can have openings and penetrations as shown in Fig. 5 with added vertical and horizontal members as boundary elements at the openings. The openings that are outside the diagonal tension and compression fields are preferred.
2. Unstiffened shear walls, in general, end-up being relatively thin. Handling large and thin steel plates during construction, especially field welding of the thin plates to the boundary columns and beams can pose some difficulty. Also, welding very thin plates, even in the shop can be somewhat difficult.
3. Unstiffened shear walls need to be checked for stiffness, to ensure that under the service design wind load the diagonal buckling of the shear wall will not occur, and the stiffness of the shear wall after buckling is sufficient to satisfy the inter-story drift limits for wind and earthquakes.
4. Unstiffened steel shear walls create relatively large bending moments, axial force, and shear in the boundary columns due to tension field forces applied to the columns, making the boundary columns quite large. The solution is to eliminate the lateral forces, which can be done by using a stiffened shear walls, composite shear walls, and the High-Performance Steel Plate Shear Wall (Qian 2017; Qian and Astaneh-Asl 2016a, 2017) where the unstiffened steel plate is not connected to the boundary columns. Also, by using slit shear walls (Cortes and Liu 2011a, b), the lateral forces applied to the columns can be reduced. More on the new high-performance steel plate shear wall and slit shear wall are given later in this chapter.

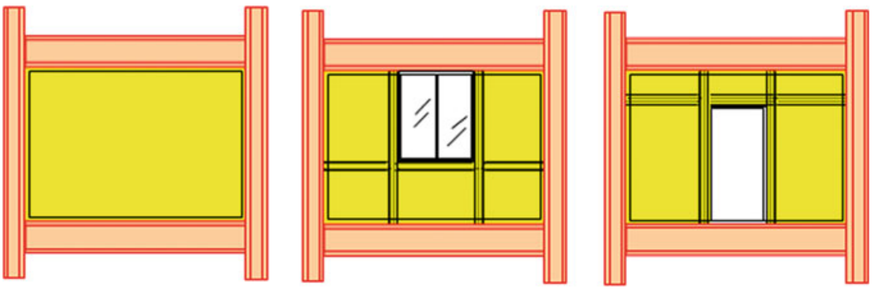


Fig. 5 Openings in steel shear walls and boundary elements around the openings

4 Examples of Constructed Steel Plate Shear Wall Buildings

Since the 1960s, stiffened steel shear walls have been used in Japan for new construction, and since the 1970s, they also found application in the U.S. in both seismic retrofit projects and new buildings. Later in the 1980s and 1990s, unstiffened steel shear walls became popular in the U.S and Canada, and recently in China. The first major tall steel plate shear wall building was the 53-story Shinjuku Nomura Building in Tokyo completed in 1978. The shear walls in this steel structure were stiffened (Astaneh-Asl 2002b). Currently, the tallest steel shear wall building in the world is the 74-story Tianjin World Financial Center in China (Sarkisian and Mathias 2012; Lee et al. 2010). More information on this structure and its design are provided later in this chapter.

In the United States, currently, the tallest steel shear wall building is the 52-story Los Angeles Convention Center Hotel (Youssef et al. 2010, 2011). Steel shear walls occasionally have been used in low rise residential building successfully to control stiffness (Eatherton 2006; Eatherton and Johnson 2004). Steel shear walls have also been used in retrofit projects. One example is the Veterans Administration Medical Center in Charleston, South Carolina. The decision to use steel shear walls instead of concrete walls was based on the need to minimize the disruption of services in the hospital, which justified the higher cost of using steel shear walls instead of concrete shear walls. The designers pointed out that wall stiffness requirements governed the design and prevented the use of thinner walls (Baldelli 1983). The stiffness of unstiffened steel shear walls can be an issue and was also mentioned in Design Guide 20 (AISC 2007). Oregon State Library is an example of retrofit of reinforced concrete moment frame with steel plate shear walls. The steel plate shear wall was chosen because book relocation could be avoided during steel construction. Bolted splices were used to minimize the risk of fire from welding in the library (Robinson and Ames 2000). Steel shear walls have also been used to strengthen steel moment frames that were damaged during the 1994 Northridge earthquake.

5 Actual Performance of Steel Shear Wall Buildings During Earthquakes

5.1 The Sylmar County Hospital (Old Olive View Medical Center), 1994 Northridge Earthquake

The Olive View Medical Center shown in Fig. 6, is a 6-story steel structure with reinforced concrete shear walls in the lower two floors and steel shear walls in the four upper floors. The floor system is a concrete slab on the steel deck. The bottom two floors have a rectangular plan, and the plan of the upper four floors is a cross

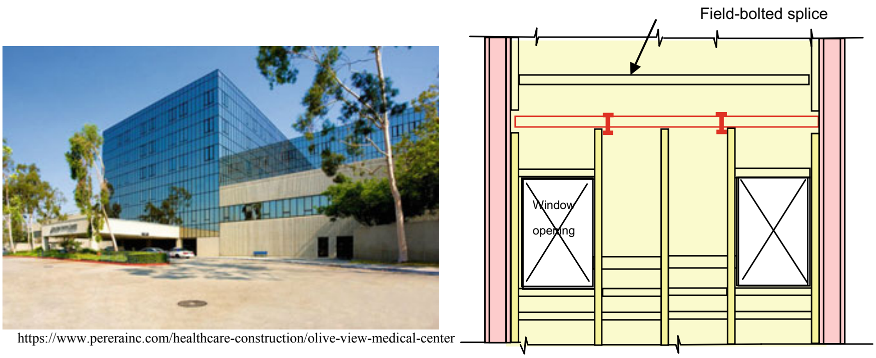


Fig. 6 The Olive View Medical Center and view of its typical shear walls

shape with stiffened steel shear walls in its perimeter (Troy and Richard 1979). During the 1994 Northridge earthquake, the building sustained serious damage to its sprinkler systems and fixed-in-place equipment, and the hospital could not function even though no damage was reported to its structure. The California Strong, Motion Instrumentation Program, instrumented the building.

The records obtained from the instrumentation indicated that on the East wall, the ground acceleration was 0.8 g, and at the roof, was 1.5 g (Astaneh-Asl 2002b). The investigation of damage to this building in the aftermath of the 1994 Northridge earthquake, by the first author, indicated that there was severe damage to some non-structural elements such as suspended ceilings and sprinkler system resulting in breakage of some sprinklers and flooding of some floors. Also, most TV sets bolted to the wall of the patient rooms had broken the connections to the wall and were thrown to the floor. The non-structural damage was an indicator of the relatively high stiffness of this structure, which was also the cause of relatively large amplification of accelerations from the ground to roof level. More information on seismic responses of this structure can be found in (Celebi 1997).

5.2 The 35-Story Office Building, 1995 Kobe Earthquake

One of the most important buildings with steel plate shear wall, subjected to a relatively strong earthquake, was the 35-story high-rise in Kobe, Japan, Fig. 7, which was subjected to the 1995 Kobe earthquake. The study of this building by Fujitani et al. (1996) indicated that the damage consisted of local buckling of the stiffened steel plate shear walls on the 26th story, Fig. 7, and a permanent roof drift of 225 mm in northerly and 35 mm in westerly directions. The results of inelastic analyses of this structure reported in Fujitani et al. (1996) indicates that soft stories may have formed at floors between 24th and 28th level of the building (Fujitani et al. 1996). A

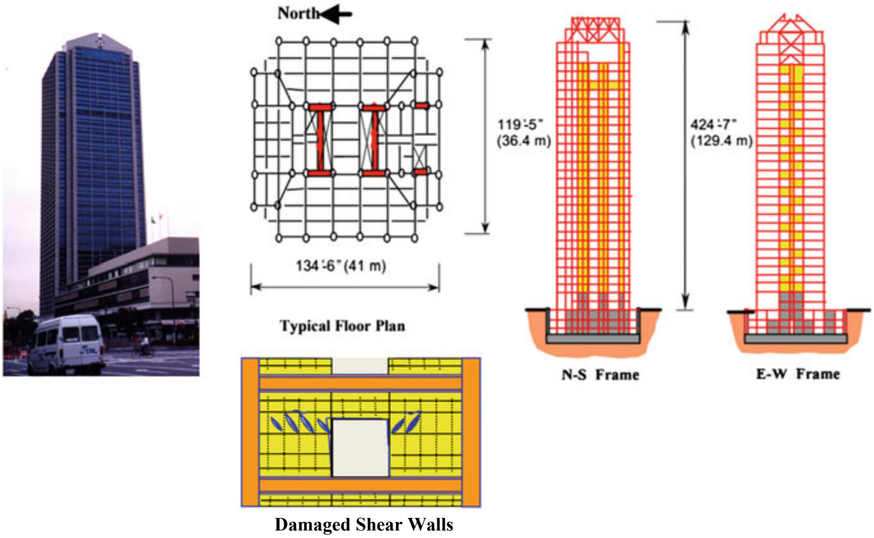


Fig. 7 The 38-story steel shear wall building and damage during the 1995 Kobe earthquake (Astaneh-Asl and Zhao 2000)

visual inspection of the structure two weeks after the earthquake by the first author did not show any sign of visual damage from outside.

6 A Brief Summary of the Past Research

Basler and Thurlimann (1961) proposed a theory for calculating the shear capacity of steel plate girders, which served as the basis of several analytical models developed later for the unstiffened steel plate shear wall system (SPSWs). Starting in the early 1980s, the post-buckling strength of steel plate shear walls was investigated at the University of Alberta, where Thorburn et al. (1983), Timler and Kulak (1983) and Tromposch and Kulak (1987) tested several single and multi-story specimens under quasi-static cyclic load, and proposed the use of a strip model to compute the post-buckling shear strength of the steel plate shear walls (Thorburn et al. 1983; Tromposch and Kulak 1987; Timler and Kulak 1983). The initial strip model proposed by Thorburn et al. (1983) was found to be capable of predicting the overall force-displacement response well but tended to overestimate the elastic stiffness. Based on test results, Timler and Kulak (1983) modified the tension field action equation proposed by Thorburn et al. (1983) for multi-story systems and included the effect of flexural stiffness of the columns.

Caccese et al. (1993) conducted tests for a series of three-story quarter-scale specimens to study the effects of different parameters on the behavior of SPSWs.

They recognized the difference in the governing limit state for thin and thick infill plates—the former is governed by yielding of infill plates, and the latter is governed by the instability of columns. Lubell (1997) tested two single panel specimens and one four-story specimen under fully reversed cyclic quasi-static loading. They found that the infill plates significantly reduced the rotational demand on the beam-to-column connections by providing a redundant lateral force resisting mechanism. The simplified strip model was again reported to be adequate in predicting post-yield strength, but not the elastic stiffness.

Driver (1997) tested a four-story-one bay specimen including gravity effects under cyclic loading. They reported that most of the energy dissipation occurred by the plates yielding with limited yielding at the beam-to-column moment connections. The strip model was found to be adequate for predicting ultimate strength but underestimated the initial stiffness. A revision of the hysteretic model proposed by Tromposch and Kulak (1987) was also proposed, in which the contributions from the moment frame and the infill panels were explicitly separated. Schumacher et al. (1999) studied four infill plate-to-boundary element connection details. The load-displacement responses of all four specimens had similar force-displacement and energy-dissipation behavior regardless of the plate-to-boundary detailing. Tears were concentrated in the corner region of three specimens.

Lubell et al. (2000) pointed out that the Canadian code provision at that time, CAN/CSA-S16.2-M94 (CSA 1994), may not be adequate for multi-story steel shear wall frames since it fails to incorporate the effects of (1) large overturning moments of multi-story frames, (2) infill panel aspect ratio and (3) the undesirable yielding sequences of the system components.

Astaneh-Asl (2002b) developed seismic design procedures for steel shear walls, which included Response Modification Factor, Displacement Amplification Factor, and Overstrength Factor. Zhao and Astaneh-Asl (2004a) tested two specimens of an innovative coupled steel shear walls system developed and used by Skilling, Ward, Magnussen, and Berkshire, now as Magnusson Klemencic Associates, a structural engineering firm in Seattle. This innovative system was used in at least one tall building. In this system, the boundary columns are relatively large concrete-filled tube composite section easily capable of resisting lateral forces of the tension field forces applied to them. Between these two boundary columns, there are two steel wide flange columns. These steel columns are not gravity columns and are allowed to yield during large seismic events. Horizontal beams are connected to all four columns. Steel shear walls are used in the two side panels. The shear walls act as coupled shear walls. The results indicated that the system has relatively high strength, high stiffness, ductility, and energy dissipation-capacities, reaching larger than 0.03-radian inter-story drift and up to 15 inelastic cycles. More information on these tests is provided later in this chapter.

Sabouri-Ghomi et al. (2005) presented a new method for calculating the shear capacity of the steel shear walls. In their method, the shear capacity of the wall was the sum of the shear capacity of the steel plate and the columns. They provided a simple, mechanics-based equation to calculate these two shear capacities and then compared the prediction of the model to available test results, showing that the predictions are quite close to test results.

The behavior of steel plate shear wall with reduced beam section connections and composite floors was studied by Qu et al. (2008) through a two-phase experimental program on a full-scale two-story specimen. The buckled panels due to progressively increasing ground motions in Phase I was replaced by new panels before applying additional shakings in Phase II. The study verified the reparability and redundancy of the SPSW such that the repaired specimen can survive a subsequent earthquake without severe boundary frame damages or overall strength degradation, and could achieve story drifts up to 5.2% (Qu et al. 2008).

Park et al. (2007) tested five single-bay three-story specimens. They found that the shear strength and energy dissipation capacity of the steel plate walls increased in proportion to the width of the infill steel plate. Different infill plate-to-boundary connections details were also studied, and the advantages and disadvantages of each alternative were discussed (Choi and Park 2009).

Researchers proposed a plastic design method for the SPSW (Berman and Bruneau 2003), and studied many other code-based design aspects of the steel plate shear walls, such as the capacity design method for the boundary column (Qu and Bruneau 2010), how to avoid in-span hinges in the boundary beams (Qu and Bruneau 2009) and the method to reduce the system over-strength by using the balanced design concept (Purba and Bruneau 2014). By using the proposed plastic and capacity design methods, it was shown that the behavior of the SPSW system could be improved.

Shi and Astaneh-Asl (2008) investigated the design of steel plate shear walls using different design philosophies. They found that the plate girder design procedures applied to steel plate shear walls can predict the shear strength of the walls reasonably well and can lead to more economical designs. The study also established that the lateral stiffness of the wall decreases considerably as the diagonal buckling of the wall occurs at relatively small lateral drifts.

Bhowmick et al. (2009) studied strain rate and P–Δ effects and found that the loading rate increases flexural demand mostly at the base of the steel plate shear wall but has limited effects on the inelastic seismic demands for a suite of spectrum compatible earthquake records for Vancouver. They also pointed out the conservative nature of the current National Building Code of Canada stability factor approach to include the P–Δ effects for steel plate shear walls, and that P–Δ has small effects on seismic demand estimations.

Two-phase pseudo-dynamic tests on a two-story steel plate shear wall system conducted by Lin et al. (2010) showed that the steel shear wall specimens could withstand three earthquakes without significant wall fracture or overall strength deterioration, but with reduced energy-dissipating capacity when subjected to the same ground motions again in Phase II. The horizontal restrainers used in Phase I design were found to be effective in improving the serviceability of the steel plate shear wall systems. The use of a strip model and equivalent brace model was also reported to be adequate in predicting global system response provided that the boundary elements are properly designed based on capacity design principle. Habashi and Alinia (2010) examined the wall frame interaction for the system. They concluded that with practical steel plate shear wall dimensions (Length/Height < 2), if the system is designed as per the AISC Design Guide 20 rules, the frame behavior is independent of the infill plate; therefore the shear capacity of the system can be calculated by simply adding frame and the infill plate capacities. Baldvins et al. (2012) proposed a set of fragility functions for steel plate shear walls for use in the performance-based design applications.

Kharmale and Ghosh (2013) proposed a performance-based plastic design method for steel plate shear walls with rigid beam-to-column connections, where a specific ductility demand and a preferred yield mechanism are chosen as the performance target. Before this work, Ghosh et al. (2009) proposed a ductility or displacement-based design methodology for SPSW systems with simple beam-to-column connections. They considered the target displacement ductility ratio and pre-selected yield mechanism with inelastic energy balance concept in the formulation. Plastic design is performed to detail the frame members and connections to achieve the target ductility ratio and yield mechanism.

Hosseinzadeh and Tehranizadeh (2014) reviewed the code-designed SPSW system and found that the boundary frames are effective in resisting story shear only in a few lower stories, while in the upper stories the bulk of the story shear is taken by the infill plates. They also found that about 70–80% of the compressive axial force in the boundary columns results from plate tension field action. Zhang and Guo (2014) established a reduction coefficient for the shear capacity of the system considering the pre-compression effect from the shortening of the boundary columns.

7 Behavior of Typical Steel Plate Shear Walls

Figure 8 shows the results of the inelastic push-over of a typical unstiffened steel plate shear wall designed according to the AISC-341 standard (AISC 2016a) and reported in the Design Guide 20 (AISC 2007).

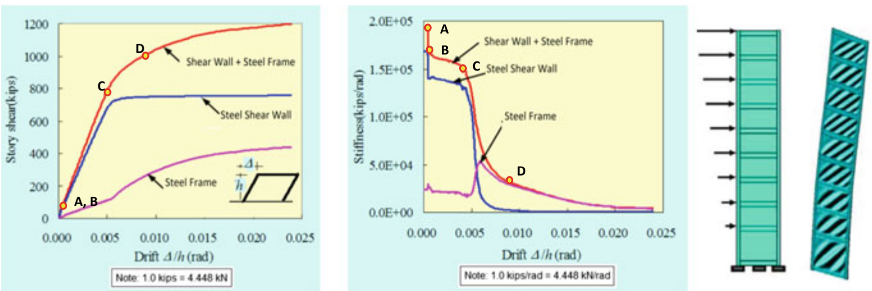


Fig. 8 Variation of story shear (left) and story lateral stiffness (right) versus story drift for a typical unstiffened single-bay steel shear wall (Shi and Astaneh-Asl 2008)

During the first phase of a pushover, the steel shear wall, and the boundary steel frame are elastic. At Point A, diagonal buckling of the unstiffened plate occurs. Due to the relatively high height-to-thickness ratio of typical steel plate shear walls, diagonal buckling occurs under relatively small story drift in the order of 0.001 radians. The shear force causing diagonal buckling was about 12% of shear yield strength of the shear wall. After diagonal buckling of the wall, lateral stiffness of the wall drops from Point A to Point B, Fig. 8 (right). The drop is about 10% of the initial stiffness of the wall. After buckling, the wall continues to resist the applied shear due to development of the tension field action. At point C the diagonal tension field yield, which causes a significant drop of lateral stiffness of the system to about 20% of the initial stiffness. At the point of yielding of the tension field area, the story drift is about 0.005 radians which is relatively small.

The lateral stiffness of steel plate shear wall designed according to the current AISC-341 standard (AISC 2016a) can be a concern and may require adding to the thickness of the wall or adding more moment frame bays as is done in Design Guide 20 (AISC 2007) Adding to the thickness of the wall will result in the development of larger tension field actions, which in turn will subject the columns to even larger lateral loads, forcing the designer to increase the size of the columns. Adding more bays of moment frames will result in higher costs since the design, construction and inspection of moment connections is quite high and can add to the cost of the structure significantly. Such solutions, which require using material and labor to increase the stiffness to satisfy the story drift limitation of the seismic codes without a need for such increase for strength purposes, can result in making the unstiffened steel shear wall uneconomical compared to other lateral force resisting systems, especially the concentrically braced frames.

As we will show later in the chapter, one solution to this problem can be to separate the steel shear wall from the columns so that the lateral forces due to tension field action do not act directly on the columns. Doing so will result in selecting the thickness of the steel shear wall to satisfy both strength and stiffness requirements without being forced to increase the size of the columns or making the shear beam-to-column connections outside the boundary frame rigid moment connections. Another solution to the problem of stiffness demand is instead of using unstiffened, use stiffened steel shear walls, where the stiffeners are designed such that the buckling of the steel panels will not occur before the yielding of the steel plate, then the system will remain elastic until the wall has yielded in shear. In this case, also, there will be no significant lateral loads acting on the columns, and the thickness of the plate can be adjusted to satisfy the strength and stiffness requirements without changing the size of the columns.

8 Modeling Steel Shear Walls

The history of analytical models and design concepts for steel shear walls dates back to the 1960s and design of steel plate girders when Basler and Thurlimann (1961), proposed analytical models and design procedures for the steel plate girders.

These plate girder design procedures served as the basis for many analytical models developed later for the steel plate shear walls.

In Japan, steel shear walls were modeled as concentrically braced frames, where the shear wall infill plates were replaced with X-bracing. During 1980s research in Canada (Tromposch and Kulak 1987; Timler and Kulak 1983; Thorburn et al. 1983) resulted in the development of the "strip" model where the steel shear wall panel was replaced with parallel diagonal truss members. The finite element models of the steel shear walls have also been used in the design of steel shear walls in important buildings such as the 73-story Jinta Building (Lee et al. 2010). In the following, the plate girder model, the shell finite element model, the strip model, and the diagonal truss model are summarized.

8.1 *The Plate Girder Model and Design Procedure*

Steel plate girders were studied during the 1960s by (Basler et al. 1960; Basler and Thuerlimann 1961) resulting in design procedures that recognized the development of diagonal buckling of the web plate, followed by the development of tension field action as the mechanisms of shear resistance in plate girders with relatively high height/thickness ratio of the web. Shi and Astaneh-Asl (2008) showed that the prediction of the shear strength of steel plate shear wall, modeled as plate girder and designed using the plate girder design equations closely matched the test results of steel shear walls, making the application of the plate girder design equations in the ASIC-360 standard (AISC 2016b) to design of steel shear walls a viable option.

Figure 9 shows the similarities between a plate girder and a steel shear wall. The columns in steel shear wall act as the flange of a plate girder and the beams in the steel shear wall act as stiffeners in a plate girder. Similar to plate girders, unstiffened steel plate shear walls under the applied load develop diagonal shear buckling at relatively small loads. After buckling, the applied shear is resisted primarily by the diagonal tension field action of the shear wall.

8.2 *The Shell Elements Model*

Currently, one of the most common methods of analysis to establish seismic forces is to model the structure elastically and subject it to "equivalent" static force established by governing codes such as the ASCE-7 (ASCE 2016). Since the analysis is elastic, the buckling phenomenon cannot be modeled directly. For design purposes, the steel shear wall can be modeled as elastic "shell" elements and the boundary beams and columns as elastic "beam" elements. The shell elements can be assigned isotropic properties to simulate the stiffness of the wall before diagonal buckling occurs. Then, a second analysis needs to be done by assigning the shell elements orthotropic

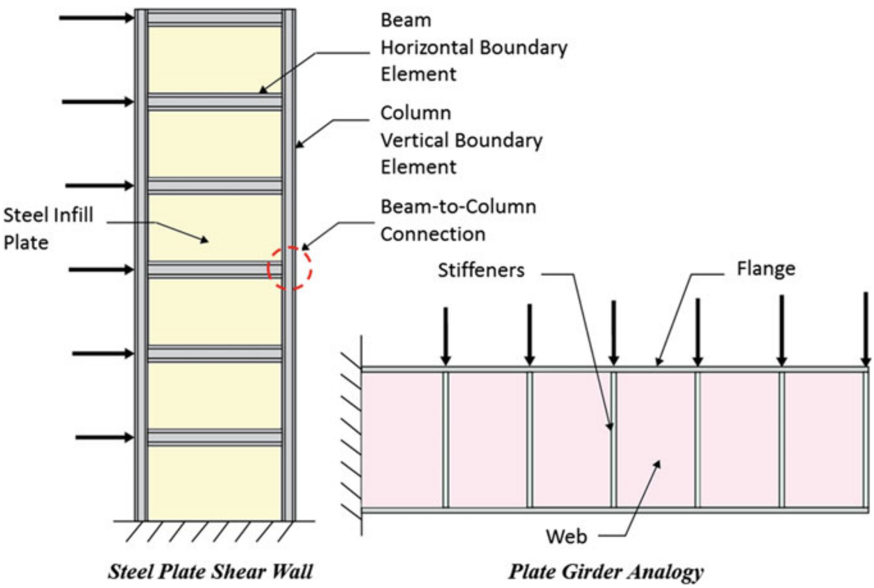


Fig. 9 Analogous relationship of the steel plate shear wall and the plate girder

properties, with the stiffness along the compression field much smaller than the stiffness along the tension field.

For important structures, such as high-rise buildings in high seismic areas, and especially when performance-based design procedures are used, (Lee et al. 2010) inelastic shell elements are used to model the steel infill plates, and inelastic beam-column members are used to model boundary beams and columns (Fig. 10 as an example). The boundary columns and beams can also be modeled using inelastic shell elements. Several past studies (Elgaaly et al. 1993; Shi and Astaneh-Asl 2008; Qian and Astaneh-Asl 2017) have shown that such “all-shell” modeling of steel shear walls predicted the stiffness and yield capacity of the shear wall, the two most important design parameters, quite accurately, although the strain hardening and failure of the first floor columns, during the late stages of the loading was not predicted by the model as accurately. Refining the material stress-strain curve and introducing initial imperfections in the columns, can improve the prediction of strain hardening and column failure.

8.3 The Strip Truss Modelling and Design

In 1980s researchers at the University of Alberta, investigated the actual behavior of steel shear wall specimens subjected to monotonic and cyclic story shear (Thorburn et al. 1983; Timler and Kulak 1983; Tromposch and Kulak 1987). They proposed

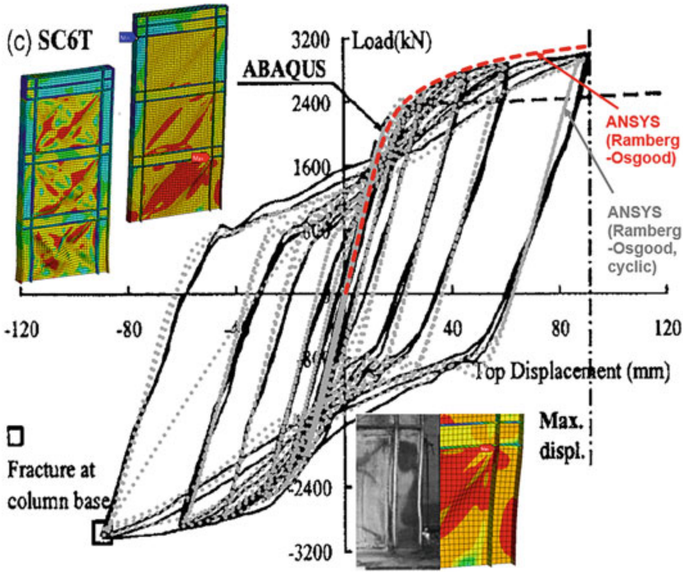
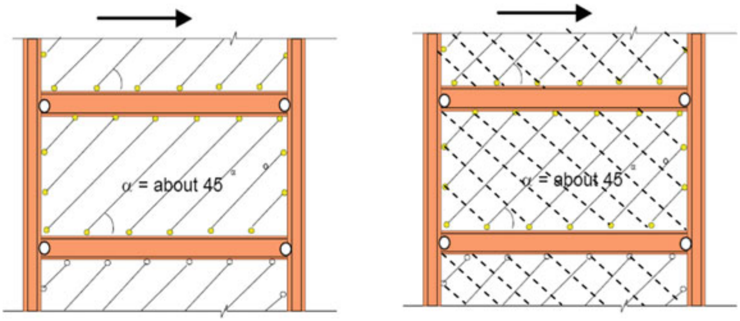


Fig. 10 Monotonic and cyclic pushover curves obtained from inelastic FE analysis compared to test results by Park et al. (2007) (Qian and Astaneh-Asl 2017)

a strip model, shown in Fig. 11a to be used for the shear wall in the analysis of the structure to establish forces developed in the components of the steel shear wall system. The walls in these studies were all unstiffened. The strip model is currently included in the North American design codes; AISC341 standard (AISC 2016a) and CAN/CSA S16-14 (CSA 2014). In this model of steel shear wall, the buckling capacity of the steel shear wall is ignored, and the steel plate is replaced by 10 or more diagonal parallel plate strips acting as pin-ended truss elements. The angle α of the strips with the vertical boundary elements, i.e., columns. The AISC-341 standard (AISC 2016a) has an equation to calculate α and also allows the angle α be taken as 40°. It was shown the strength of SPSW designed in compliance with current AISC requirements is not substantially sensitive to the angle of inclination and using a value of 40° will generally lead to slightly conservative results (Dastfan and Driver 2008).

The strip model represents the post-buckling condition of the wall and ignores the diagonal compression capacity of the wall. The model is reasonable for predicting maximum forces in the diagonal tension field as well as the lateral forces applied to boundary elements only in relatively slender walls. However, by ignoring the strength and stiffness of the compression field, the model underestimates the lateral stiffness of the shear wall system. Since seismic forces, in reality, are dynamic forces, underestimating stiffness of a system subjected to dynamic forces, will result in underestimating the actual dynamic forces generated in the system, especially in tall buildings, where the walls are relatively thick in lower floors, and their strength and stiffness in diagonal compression field cannot be ignored.



(a) Strip Model in the Current North American Design Codes

(b) Strip Model in the Current North American Design Codes

Fig. 11 The strip model of the steel shear wall

As Fig. 3 earlier shows, for unstiffened steel shear walls with height/thickness ratio of 300 or more, the shear buckling stress is 10% or less of the shear yield stress, therefore, conservatively can be ignored. A modification of the strip model is shown in Fig. 11b, where diagonal compression strips are also added. This model can correct the shortcoming of the strip-model for walls with a height/thickness ratio of more than 300. The width of compression strips can be equal to the width of tension strips times the ratio of shear buckling stress to shear yield stress given in Fig. 3 earlier.

Topkaya and Atasoy (2009) studied pre- and post-buckling shear stiffness of steel shear walls using nonlinear finite element analysis of the wall as well as the results obtained from the strip model. They concluded that (a) the lateral stiffness obtained from the finite element method is higher than the test results and can be considered an upper bound solution; (b) the strip method of analysis offers less stiffness than the test results and can be considered as a lower bound solution. Topkaya and Atasoy (2009) proposed an equation to establish pre- and post-buckling stiffness of the unstiffened steel shear walls.

8.4 The Diagonal Truss Model

The equivalent brace (EB) model shown in Fig. 12 was first proposed and recommended by Thorburn et al. (1983) for preliminary design purposes and is included in the Canadian standard CAN/CSA-S16. The inherent assumption of EB model is that all tension strips develop the same strain and can be replaced by a single diagonal truss member. The properties of the equivalent brace are related to the properties of the infill plate by matching the shear strength and stiffness of the equivalent brace to the original steel shear wall. Due to its simplicity, the EB model has also been commonly used in the analysis of SPSWs. However, certain phenomena cannot be captured by the EB model such as the tension field effects on the columns and the

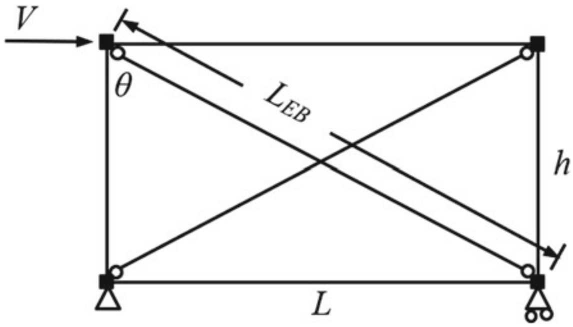


Fig. 12 The equivalent brace model of the steel shear wall

compression resistance of the panel near the corners. These concerns will be less of an issue if the infill panels are connected to the beam only since the column is no longer subjected to the distributed lateral tension field forces.

The brace area and yield stress for conventional unstiffened steel plate shear walls are shown in Eqs. 1 and 2.

$$A_{EB} = \frac{t_p L}{2} \frac{\sin^2(2\alpha)}{\sin\theta \sin(2\theta)} \tag{1}$$

$$f_{yEB} = \frac{\sin(2\theta)}{\sin(2\alpha)} \frac{L_{cf}}{L} f_{yp} \tag{2}$$

where α is the angle of inclination defined earlier, which can be taken as 40°, θ is the angle between the diagonal brace and columns; t_p is the infill plate thickness, and f_{yp} is the minimum specified yielding stress of infill plate. L is the center to center distance between columns, L_{cf} is the clear distance between column faces, and h is story height.

9 Design of Steel Shear Walls

The steel shear walls currently covered by design codes such as the AISC-341 standard (AISC 2016a) and called Special Plate Shear Walls (SPSW) consists of unstiffened steel *infill plates* connected to the *boundary steel moment frame*. The design procedures for the SPSW in the AISC-341 standard (AISC 2016a) are formulated to make the system to behave in a ductile manner under cyclic lateral forces by primarily shear yielding of the steel infill plates and development of plastic hinges at the ends of the horizontal boundary elements, i.e., beams. The boundary columns and all connections in the system are expected to remain essentially elastic. To achieve the desirable ductile behavior, the unstiffened infill plates are designed to yield in shear under the maximum applied shear force. Since the infill plates are quite slender, they buckle diagonally during early stages of application of the shear and develop diagonal tension field action, which resists the applied shear.

9.1 Design of the Infill Plate Using Plate Girder Equations

The infill plates in early steel shear walls were designed using equations developed for steel plate girders and given in design specifications such as the AISC-341 standard (AISC 2016a). Shi and Astaneh-Asl (2008) took the 9-story steel shear walls given in the design example in AISC Design Guide 20 (AISC 2007) and redesigned them using the steel plate girder equations. The shear walls in the AISC Design Guide were designed using the "Strip" model procedures currently in the AISC-341 standard (AISC 2016a). Figure 13 shows the two design cases. Case 1A is the frame included in the AISC Design Guide 20 (AISC 2007) and designed following the procedures in the current AISC-341 standard (AISC 2016a).

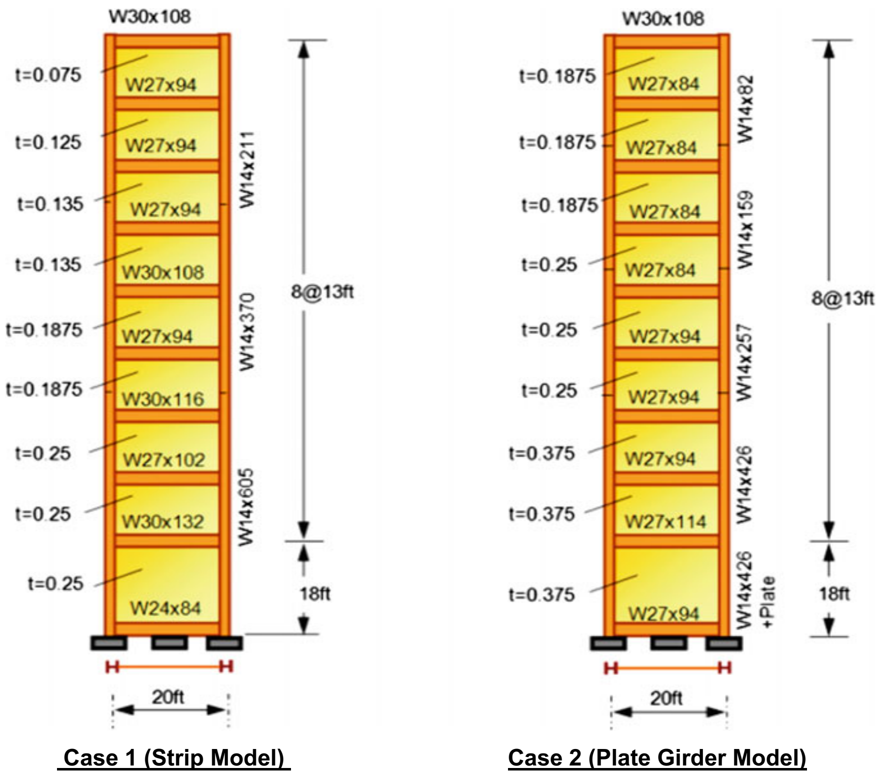


Fig. 13 Shear wall designed using "strip model" (left), and plate girder model (right) (Shi and Astaneh-Asl 2008)

Case 2A is designed following the procedures proposed by Astaneh-Asl and Shi (2008). The main difference between these two cases is that the columns in Case 1A are much heavier than the Columns in Case 2A while the walls in Case 1A are much thinner than the walls in Case 2A. The beams in Case 1A are also heavier than the beams in Case 2A. The infill wall thickness for Case 1A was less than the wall thickness for Case 2A. In both cases, all beam-to-column connections are moment connections. The weight of the 9-story shear wall and boundary columns and beams for Case 2A, designed using plate girder equations was 85% of the weight of Case 1A, which was designed using the strip model currently in the AISC-341 standard (AISC 2016a). Since the fabrication costs for both cases were almost the same, the comparison of the weight is a good measure of construction costs savings. By using the plate girder equations to design the shear wall, we could save about 15% in the cost; now, the question is what the performance of these two designs under lateral forces.

Figure 14a shows the variation of stiffness versus story drift for the shear wall in the 4th floor of both designs. The design Case 1A, the Strip Model design, shows higher initial stiffness until the infill plate buckles. The slightly high initial stiffness is due to the stiffness of relatively heavy beams and columns in the boundary frame. However, both design cases buckle diagonally at a drift level of about 0.0007. As mentioned before, diagonal buckling of the unstiffened shear wall under such relatively small drift values is a concern that is not addressed in the current AISC-341 standard (AISC 2016a). After buckling of the diagonal, tension field action develops and continues to resist shear. Case 1 design based on the current Strip Model and procedures in the AISC-341 standard (AISC 2016a) yields under a story drift of about 0.004 and drops the lateral stiffness abruptly, while the design based on the plate girder model yields at 0.0055 story drift and drop of the lateral stiffness is more gradual. Figure 14b shows the variation of story shear versus story drift. Even though, Case 2A design, based on the "Plate Girder Model" was 15% lighter than the design using the strip model, the shear yield capacity of the Case 2 shear wall was about 25% higher than

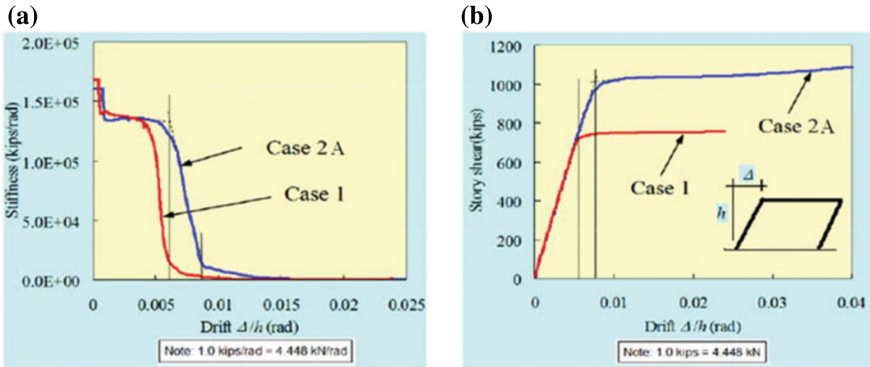


Fig. 14 **a** Lateral stiffness versus story drift, and **b** story shear versus story drift

the shear wall designed in AISC Design Guide 20 (AISC 2007) using the AISC (2016a) strip model procedures.

The studies by Shi and Astaneh-Asl (2008) summarized above indicated that using the plate girder equations to design steel shear walls results in a system that behaves better than a system designed using the strip model and provisions in the current AISC Seismic Provisions (AISC 2016a), has about 12% higher yield strength and is 15% lighter than the system designed using strip model.

9.2 Design of the Infill Plate Using the Strip Model

In this approach, which is currently in the AISC-341 standard (AISC 2016a), the nominal shear strength of the Special Plate Shear Walls is established by the following equation, using only tension field action and ignoring shear buckling capacity of the infill panel.

$$V_n = 0.42 f_{yp} t_p L_{cf} \sin 2\alpha \tag{3}$$

where α is the angle of infill plate yielding measured relative to the vertical defined by Eq. 3 given earlier, f_{yp} is the specified minimum yield stress of infill plate, t_{p} is the thickness of the infill plate, and L_{cf} is the clear distance between column flanges. In this method, there are limitations on the moment of inertia of the boundary columns and beams. The moment of inertia of the boundary column should be larger than 0.00307 $t_{\mathrm{w}}h^4/L$. A ϕ-factor of 0.90 is used in Load and Resistance Factor (LRFD) to compare the above shear strength to the applied shear strength resulting from the application of factored lateral forces to the system.

While the plate girder equations can be applied to all values of depth-to-thickness ratio, the strip model is developed using experimental data on highly slender shear walls. It is suggested that the use of the strip model should be limited to shear walls with a height-to-depth ratio more than 500. For walls with h/t less than 500, the plate girder procedure can be used or a realistic inelastic finite element analysis or testing be conducted to establish their actual behavior.

9.3 Design of the Infill Plate Using Finite Element Model

In this method of analysis of shear wall, the infill plate is modeled as elastic or inelastic shell elements. If elastic shell elements are used, to incorporate the diagonal buckling of the infill plate, the material of the plate can be orthotropic and rotated 45° with respect to the beams. The modulus of elasticity of the diagonal tension direction can be the full elastic modulus, and for the compression diagonal direction, the modulus of elasticity can be 10% of the full modulus of elasticity.

(Photo courtesy of SOM)

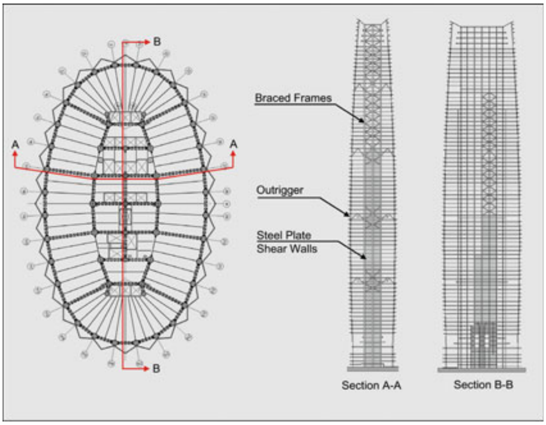


Fig. 15 The Tianjin World Financial Center and its structural system (Lee et al. 2010)

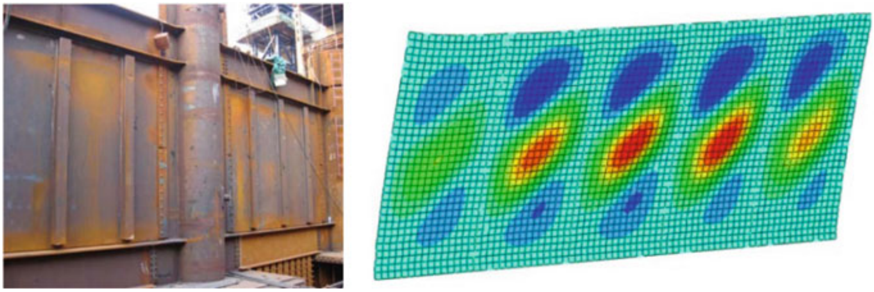
In the past decade, two high-rise buildings have been designed and constructed using steel plate shear walls. One is the 74-story Tianjin World Financial Center in China, which has vertically stiffened steel shear walls. The other steel shear wall tall building is the 55-story Los Angeles Convention Center Hotel, which has unstiffened steel plate shear walls (Youssef et al. 2010). Both have heavily used finite element methods in the analysis of the steel shear walls. In the following design procedures used in the design of the 74-story Tianjin Building is summarized. The building is located in high seismic and high wind area.

Design procedures used in the design of 74-story Tianjin World Financial Center

Figure 15 shows a view of the Tianjin-WFC building, its framing plan and elevation view of its typical shear wall frames. The steel shear walls in this building are vertically stiffened outrigger, steel shear walls for lower 2/3 of the height and outrigger concentrically braced frames for the top 1/3 of the height. The columns are concrete-filled composite round tube members and the beams are I-shaped girders acting compositely with the floors. All beam-to-column connections in steel shear wall system are rigid moment connections.

The designers (Lee et al. 2010) state that in this case, they studied other options such as concrete shear walls, composite shear walls, and dual concentrically braced frames. The concrete shear walls were ruled out because of the large dimensions of the concrete walls taking too much of occupiable floor area. The composite shear walls were not selected since the designers felt there would be a need for a significant amount of cost and time to be spent to conduct testing and research to establish the performance. The dual braced frame system proved to require 20–25% more steel than the steel shear walls to satisfy the performance requirements.

Figure 16 shows a view of the shear walls in the Tianjin WFC building and the von Mises stresses acting on a typical infill panel. In this case, the vertical stiffeners



(a) Buckling restrained SPSW : channel stiffener with a gap between end of stiffeners to the boundary beams by Lee et al. (2010)

Fig. 16 Steel shear walls of the Tianjin WFC building and von Mises stresses (Lee et al. 2010)

were designed to prevent buckling of the infill panels under the code design wind load and the "frequent" earthquakes.

The general design philosophy for steel plate shear walls in this building was established to satisfy the requirement of the Chinese code JGJ 99-98, which requires that the steel plates be designed not to buckle under frequent earthquakes and winds and have the strength to resist the strong earthquakes without exceeding the limit of inter-story drift. For the strong earthquake level, buckling of the steel plates and the utilization of tension field action is allowed. Following is a summary of main design considerations and limitations in the design of steel plate shear walls as given in Lee et al. (2010).

- The structure was designed to satisfy the Chinese code JGJ 99-98 for gravity, wind, and earthquakes with regards to strength as well as stiffness.
- The structure was designed to carry gravity loads without the contribution of the shear walls.
- The code design procedures utilize 50-year return wind and seismic loads. The seismic event corresponding to this recurrence period is termed the "frequent earthquake."
- For wind effects, the building was designed to satisfy the code drift requirement of 1/400 for 50-year wind (63.5% probability of exceedance in 50 years) which corresponded to a basic wind pressure 0.5 kN/m^2 in Tianjin, and strength requirements under the 100-year wind (basic wind pressure 0.6 kN/m^2 in Tianjin). Damping ratio was set at 3.5% considering the composite effect of the CFT columns. The building was also designed to satisfy the code wind acceleration perception requirements based on a 10-year return event (basic wind pressure 0.3 kN/m^2 in Tianjin) with damping ratio set at 1.5% by code, once again considering the CFT columns. Acceleration was limited by code to 0.28 m/s^2 at the highest occupied floor. Wind tunnel testing was required. By local practice, wind speeds used in the tests were at least as high as those stipulated in the codes in the predominant wind direction, but directional effects were permitted to be considered.

- The tower is located in a high seismic area, with a peak ground acceleration of 0.15 g specified in the local code for this site. Inter-story drift was 1/300 for this seismic event. A damping ratio of 3.5% was used. Also, the codes required that time-history analysis of the building be performed using two recorded and one simulated site-specific ground motions.
- Cyclic tests of scaled steel shear wall specimens were conducted to establish actual behavior of specimens.
- The steel shear walls were designed to remain elastic and not to buckle during the frequent earthquake and design wind loads. The steel shear walls were also designed to be the primary lateral load resisting system during moderate or rare earthquakes. The thickness of the infill plate satisfied the strength requirements of the Chinese code as well as the AISC-341 standard (AISC 2016a).
- For the elastic analysis, the infill panel was modeled using full isotropic shell elements. For inelastic time history analysis, a dual strip model, shown in Fig. 11b earlier was used.
- The boundary beams and columns of the steel shear walls were designed for the forces determined from elastic analyses to satisfy the Chinese code provisions.
- Development of plastic hinges at the end of boundary beams was allowed at the moderate and rare earthquake demand levels.
- As per the requirement of the seismic experts, the first author is one of them, some minor yielding but no plastic hinging was allowed in the boundary columns at moderate earthquake levels, and, in the lower 16 stories, some minor yielding but no plastic hinging was allowed in the boundary columns at rare earthquake levels.
- Because of the newness of the structural system, as the Chinese code requires, at the end of the design process, the seismic and wind performance of the building were reviewed by a panel of experts and additional requirements were imposed on the design.

10 Material Used in Steel Shear Wall Systems

The material of steel shear wall is usually ASTM A36 steel, with a specified minimum yield stress of 36 ksi (248 MPa), which was used in most of the steel shear wall specimens tested in the past by researchers. The use of higher grade of steel is not prohibited by seismic design codes. However, the use of high strength steel plate can result in reduced ductility of steel shear wall systems. The lateral stiffness of steel shear wall is directly proportional to the thickness of the infill plate (Thorburn et al. 1983; Sabouri-Ghomi et al. 2005), and by using higher strength steel and the associated reduced thickness, the stiffness of the shear wall will be reduced.

As mentioned under Construction Issues later, one of the problems with the fabrication of steel shear wall systems is handling and welding thin steel plate shear walls. Using higher strength steel and having even thinner walls can create problems with handling and welding in the field. Analytical studies and cyclic tests of steel stiffened and unstiffened shear walls using low yield steel has shown very ductile and desirable performance.

Chen and Jhang (2011) conducted cyclic testing of one- and two-story steel shear walls and concluded that the specimens could tolerate cyclic lateral story drifts exceeding 0.05 radians. The test specimens had rigid as well as pin connections between the beams and columns of the boundary frames. They concluded that the specimens with shear (pin) connections had 28% less strength and 18% less energy dissipation capacity than the specimens with moment connections and recommended the use of moment connections in steel shear walls with low yield steel. However, considering the current relatively high cost of design, fabrication, and inspection of the field-welded moment connections compared to shear connections, it seems that the above reduction in the strength and energy dissipation can be easily compensated for by either adding to the length or thickness of the steel shear wall itself and using shear connections instead of moment beam-to-column connections.

To satisfy strong column-weak beam design requirement, the columns in the boundary frame are usually ASTM A913 Grade 65 or ASTM A992 Grade 65 with a minimum specified yield stress of 448.5 MPa (65 ksi). The beams in the boundary frame are usually ASTM A992 Grade 50, with a minimum specified yield stress of 345 MPa (50 ksi). Using grade 65 steel for columns and grade 50 steel for beams results in satisfying the "strong column-weak beam" requirement for the boundary frame easier.

In unstiffened steel shear walls, relatively large lateral forces act on the boundary columns creating relatively large bending moments in the boundary columns. One of the efficient ways to resist such bending moments is the use of the concrete-filled tube or built-up box sections as the boundary columns. The columns of the 73-story and 55-story steel shear wall building shown in Fig. 15 are concrete-filled tube and built-up box sections respectively.

11 Design of Members and Connections

The boundary columns and beams in a steel shear wall are capacity-designed to resist gravity alone and gravity plus seismic or wind load combinations. The seismic load combinations should include *overstrength* load equal to strain-hardened, expected yield strength of the tension filed equal to 1.1 $R_y F_y t_w$. The width-to-thickness, the *b*/*t* ratios of the elements of the cross sections of the boundary beams and columns should be such that these members do not develop local buckling under cyclic loading until they yield. The AISC-341 standard (AISC 2016a) designates such members as *highly ductile members* and provides the limiting values of the *b*/*t* ratio in its Table D1.1.

For the connections, like other steel systems, the type and detailing of the connections of steel shear walls play a major role in the cyclic behavior of the system. The main connections of steel shear wall system are infill plate-to-boundary element connections, splices in the steel plate and columns, beam to column connections, the column base connections, floor to girder connections and the connections of the collectors to the steel shear wall system. The following sections provide information on the design, detailing, and construction aspects of these connections.

11.1 Infill Plate-to-Boundary Element Connection

The infill steel plate is connected to the boundary either by welds, bolts or their combination. The elements of the connection are designed following capacity design concepts to develop the strength of the steel plate using the expected yield strength of the infill plate $R_y F_y t_w$.

In fully welded option, "fin" plates are fillet-welded to the columns and beams in the shop. Then, the steel infill plate is welded to the fin plates with continuous fillet welds in the field. This option is used quite often, and almost all cyclic test specimens of steel shear walls have used this detail validating its desirable performance. In welded-bolted option, the fin plates are still welded to the boundary elements, but the infill plate is bolted to the fin plates by one or if needed more rows of bolts. In bolted-bolted detail, instead of fin plates, a tee section or double-angle section can be used, where the flange of the tee or the outstanding legs of the double-angle are bolted to flanges of the boundary columns and beams, and the stem of the tee or the back-to-back legs of the double angles are bolted to the steel infill plate. The bolted-bolted detail seems to be a viable option for stiffened shear walls where negligible tension field action forces are applied to the columns. In using this detail in unstiffened steel shear walls, the prying action on the bolts on the column flange needs to be considered in the design. No experimental results on the actual behavior of bolted-bolted connections of the infill plate-to-boundary element could be located at this writing.

Past cyclic tests of the unstiffened steel shear walls (Timler and Kulak 1983) have shown that during the cyclic loading the corners of the wall plate fractured due to the relatively large cyclic strain concentrations at the corners. To prevent the corner fracture, researchers have recommended adding welded strap plates to fill the gap between the horizontal and vertical fin plate (Tromposch and Kulak 1987), to provide corner cut-outs (Schumacher et al. 1999), or to incorporate special corner brackets and connecting the horizontal and vertical fin plates away from the corner (Choi and Park 2008). It appears that adding corner brackets can be an effective and economical solution, which was done for the 73-story Tianjin-WFC building.

11.2 Splices in the Infill Plate

In many applications, the size of the infill panel is such that the steel infill plates need to be spliced. The splices typically consist of shop-welding steel plates directly to each other or field-bolted using two splice plates one on each side of the infill plate. Figure 17 shows a view of a typical bolted splice. The splices of steel plate shear

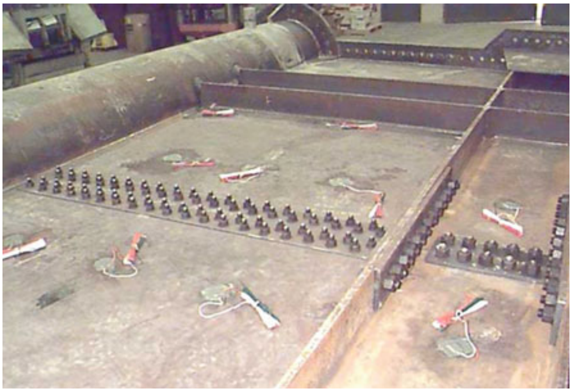


Fig. 17 A bolted steel plate splice

walls need to be designed for the strain-hardened expected yield strength of the wall equal to 1.1 $R_y F_y t_w$.

11.3 Beam-to-Column Connections

Beam-to-column connections in steel shear walls can be rigid, partially restrained (i.e., semi-rigid), or simple shear connections. The current AISC-341 standard (AISC 2016a) specifies the use of rigid moment connections but does not specify whether the connections should be *Ordinary, Intermediate,* or *Special.* In the Commentary part of the AISC-341 standard (AISC 2016a), it is mentioned that the moment connections are *Intermediate.* For more details on the beam-to-column connections design requirements and definition of the Intermediate Moment Frames; the readers are referred to the provisions of the AISC-341 standard (AISC 2016a). The seismic provisions ensure minimal ductility in the connection to enable the girders to develop plastic moment and to prevent brittle fracture in the connection area when steel shear wall undergoes large cyclic story drifts.

Cyclic tests done by Tromposch and Kulak (1987) showed that steel shear wall specimen with a fully restrained beam-to-column connection could dissipate as much as three times more energy than the system using similar frames but with bolted simple connections. The tests showed a robust rotation capacity of the bolted simple beam-to-column shear connections. Caccese et al. (1993) found that the rigidity of beam-to-column connections had a minor effect on the overall load-displacement behavior of the system. Sabouri-Ghomi and Gholhaki (2008) conducted cyclic tests of steel shear walls with the fully restrained and simple shear connection. Both specimens were able to reach story drift of 2.7%, where the shear wall developed corner tearing, and the tests stopped (Sabouri-Ghomi and Gholhaki 2008).

In their studies summarized in Shi and Astaneh-Asl (2008), the researchers conducted nonlinear push-over analyses of two steel shear walls to compare the behavior

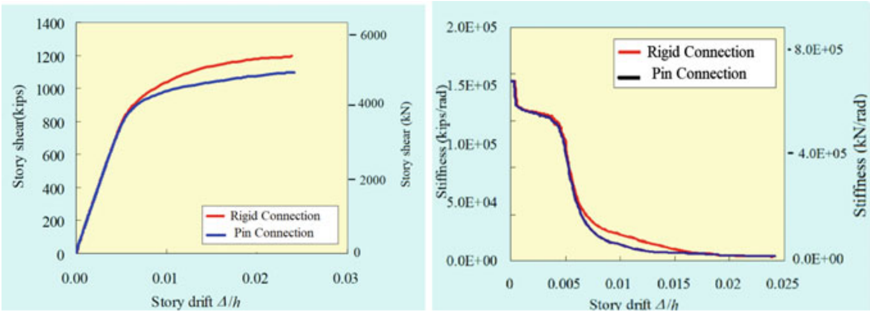


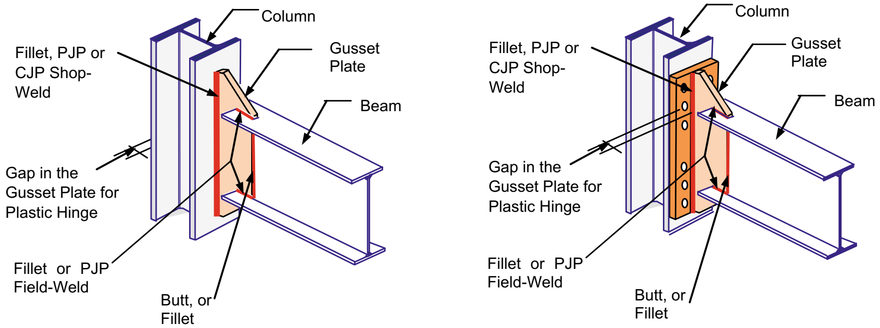
Fig. 18 Variations of story shear and lateral stiffness versus story drift for frames with rigid and pin connections

of steel shear walls with rigid and pin beam-to-column connections. One frame was the 9-story steel shear wall in the AISC Design Guide 20 (AISC 2007), which has rigid moment connections and the other was the same frame, but the beam-to-column connections were pin connections. Figure 18 shows the variation of the lateral stiffness versus story drift and variation of story shear versus story drift curves for the two frames. Both frames, one with the moment and the other with pin connections showed almost the same initial stiffness and yield point. After yielding of the shear wall; the frame with rigid connection showed more strain hardening.

Qu et al. (2008) and Vian et al. (2009) based on their studies of shear walls have recommended the use of Special Moment Connections in the steel shear walls. Higher ductility and energy-dissipation capacity of special moment connections is a desirable property; however, the beam-to-column connections are not the main source of energy-dissipation in a steel shear wall system and the rotational demand on the connections of steel shear walls are less than the demand in moment frames. The use of simple shear connection (Xue and Lu 1994; Cortes and Liu 2011b) and bolted plate moment connection (Vatansever and Yardimci 2011; Caccese et al. 1993) in the SPSW system should be re-visited, and development of new beam-to-column connections that fit the steel shear wall system should be encouraged. The potential savings gained by replacing a special or an intermediate moment connection by some other simpler and more cost-efficient beam-to-column connection need to be considered.

One of the innovative moment connections developed recently by Qian and Astaneh-Asl (2017) is the Gusset Plate Moment Connection (GPMC). Figure 19 shows the main elements of the new Gusset Plate Moment Connection (GPMC). The connection has two versions: welded and bolted.

The gusset plate in the new Gusset Plate Moment Connection is a vertical flat plate in the plane of the column and beam web. The gusset plate is mainly subjected to bending and shear, and a small amount of axial force. The plastic hinge formation is expected to occur in the gusset plate primarily due to in-plane yielding of the gusset plate. The gusset plate is designed to yield within a specified area—the gap between



Welded Version of the New Gusset Plate Moment Connection

Bolted Version of the New Gusset Plate Moment Connection

Fig. 19 Welded (left), and bolted (right) versions of the new Gusset Plate Moment Connection (Qian and Astaneh-Asl 2017)

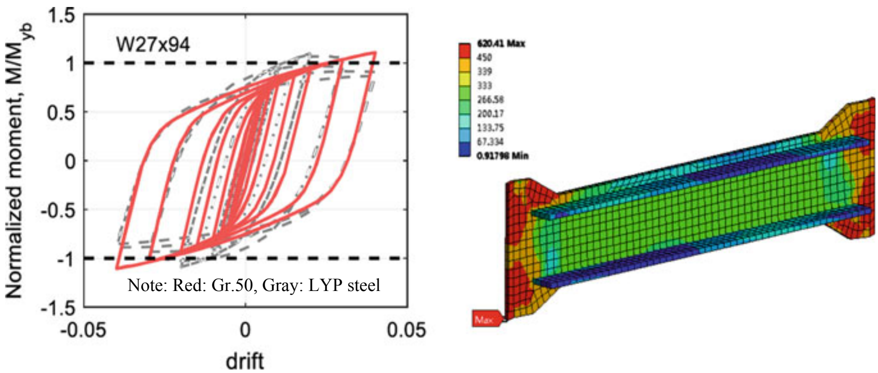


Fig. 20 Hysteresis curves (left) and von Mises stresses for a Gusset Plate Moment Connection

the face of the column flange and end of the beam, Fig. 19—and is the only inelastic element of the connection. The gusset plate, being the only element of the connection that yields and dissipates energy, acts as a "fuse" and protects all other elements of the joint such as the beam, column, welds and bolts, which are designed to remain essentially elastic during the seismic event. More information on the behavior and design of new Gusset Plate Moment Connection is in Qian and Astaneh-Asl (2017).

Figure 20 shows the cyclic behavior of a gusset Plate Moment Connection with gusset plate made of grade 50 steel (Fy = 345 MPa or 50 ksi) and low yield steel (F_y = 131 MPa or 19 ksi).

The new connection utilizes highly ductile gusset plates to provide the necessary bending strength, rotational stiffness, required plastic rotation, and sufficient energy dissipation capacity. Based on the extensive analytical studies performed, the new connection has proven to be highly ductile, easy to fabricate, and potentially cost-effective. The proposed connection has a wide range of applications in steel and composite moment frames, and composite and steel shear walls (such as the new *High Performance Steel Plate Shear Wall* discussed later in this chapter), as well as in dual steel systems composed of special concentrically braced frames and moment frames. It can be used for seismic as well as wind and gravity resistance. The most important advantage over the field-welded moment connections used in the current steel plate shear wall is that the new connection does not incorporate Complete Joint Penetration (CJP) field-welds, which not only are relatively expensive to fabricate but require costly field-inspection using ultrasonic testing equipment and expert operators. Another Advantage is that using this connection the *strong column-weak beam design* is no longer required, see Qian and Astaneh-Asl (2017).

11.4 Column Splices

Column splices in steel shear wall systems should be designed to resist at least 50% of the bending strength of the smaller of the column section being spliced AISC-341 standard (AISC 2016a). The nominal shear strength of the splice should be equal or greater than the $\Sigma M_{pc}/H_c$ where ΣM_{pc} is the plastic moment capacity of the cross sections of the columns above and below the splice, and H_c is the clear height of the column between beam-to-column connections. If welds are used in the splice, they should be complete joint penetration groove welds AISC-341 standard (AISC 2016a).

11.5 Collectors Connections

Collectors are the girders connected to the sides of the steel shear wall columns to transfer the inertia forces from the floors to the walls. The main force to be transferred is an axial force in the connector. The connection should be designed to have large axial force capacity but relatively small bending moment capacity so that the boundary columns are not subjected to additional bending. Details suggested by Astaneh-Asl (2008) for connections of collectors to the columns of concentrically braced frames can also be used for steel shear wall columns.

11.6 Column Base Connections

Columns in steel shear walls should remain essentially elastic. The only exception is that they can develop plastic hinges at their base above the base plate. The base connections of the boundary columns should be designed to transfer combined axial and bending as well as shear strength of the plastic hinge formed at the base of the column. The yield stress of the column used in establishing the axial, bending and shear strength of the column should be 1.1 R_yF_y. It is recommended that the transfer of shear from the base plate to the foundation be done by using shear keys under the base plate instead of anchor rods. The role of anchor rods should be to resist tension due to overturning and uplift of the base plates. The grout under the base plate should be high strength grout, not thicker than 2-in. (50 mm) and preferably be fiber-reinforced concrete to prevent cracking of the grout under compression as was observed in the thick unreinforced grouts under the base plates of the Northridge State University Library during the 1994 Northridge earthquake. In this case, eventually the library had to be demolished and rebuilt since unlike damage to other elements of a steel structure, that in most cases can be easily repaired, the damage to the grout under a base plate, or even damage to a base plate or anchor rods is very difficult and costly to repair.

11.7 The Connection of Infill Plate to the Foundation

The connection of steel shear wall to the foundation should be capable of resisting the shear force in the infill plate connected to the foundation. The AISC-341 standard (AISC 2016a) does not provide specific provisions on what should be the required strength of the base connection of the steel infill plate. However, applying the provisions for the fin-plates connecting the infill plate to the boundary beams and columns, the base connection of the infill plate, should be designed to resist tensile yield strength of the tension field calculated using *expected yield* strength of the infill plate, R_yF_y of the steel infill plate. Due to the importance of the base connection of the infill plate to the foundation, the authors recommend that the connection be designed to resist the tensile yield capacity of the tension field using *strain-hardened, expected yield strength,* 1.1 R_yF_y.

The base beam is optional. If only shear studs are used, then the base beam is allowed to bend upward due to tension field action and should be capacity-designed to resist the *expected yield strength* of the infill plate. If anchor rods are used and designed to resist the expected yield strength of the infill plate, then the base of the shear wall can be considered fixed.

Figure 21 shows examples of details of the connection of the steel infill plate to the foundation.

12 Construction Considerations

Currently, to construct steel shear walls, first, the boundary columns and beams are fabricated in the shop and then erected in the filed using field-welded and bolted moment connections. The steel infill plates are then welded to the fin plates on the boundary beams and columns using fillet welds. Due to the relatively thin character of the unstiffened plates, an attempt should be made to reduce the gravity loads supported by the unstiffened steel shear walls. Gravity loads can cause buckling of the thin unstiffened shear walls even during construction. In the design stage, an attempt should be made to put as little floor gravity load as possible on the horizontal boundary elements of the shear wall system. Even if the steel shear wall is not directly subjected to gravity loads, in highrise buildings, the possibility of column shortening causing buckling of the unstiffened steel shear walls should be investigated.

To prevent buckling of unstiffened steel shear walls, the top of the steel plate can be left unwelded to the horizontal beam and the welding be done after several floors above the weld line are constructed. By doing this the effects of floor gravity load, as well as column shortening in pushing down the thin steel plate, will be significantly reduced.

The potential problem of burning of the infill plate during the upward field welding has been reported (Eatherton 2006). Difficulties were also encountered in constructing the infill plate-to-column connection at the column splice location. Sometimes, relatively large holes had to be cut in the infill plates to provide space and access to the column splice connection (Youssef et al. 2011).

Steel shear walls can be prefabricated in modules in the shop and assembled in the field. Zhao and Astaneh-Asl (2004a) tested a shop-welded, field-bolted modular steel shear wall system, which was used in a 24-story building and proved to be a viable option. Driver and Moghimi (2011) investigated the modular construction of steel shear walls and proposed three different concepts. The concept of mid-height continuous splices, similar to that shown in Fig. 17, and tested by Zhao and Astaneh-Asl (2004a) was considered one of the most promising configurations regarding practicality and economics.

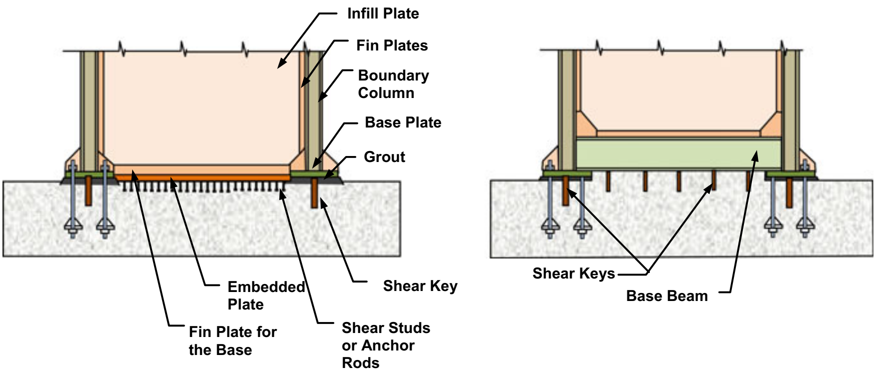


Fig. 21 Suggested details for the base connection of steel shear walls

13 Recent New Developments

The current code procedure for the design of steel shear walls utilizes the post-buckling tension field action. The use of tension field action results in the relatively large lateral and axial forces applied to the boundary beams and especially to the boundary columns. For columns, such lateral forces are applied only from one side, the infill plate side, resulting in relatively large bending moments in the columns. Such large lateral forces applied to the column make the boundary columns quite large and costly. In recent years, a few attempts have been made to develop new and more efficient steel shear wall systems. Two attempts focused on decreasing the lateral loads applied to the boundary columns (Corts and Liu 2011a, b; Qian and Astaneh-Asl 2016, 2017). The new High Performance Steel Plate Shear Wall system, developed by Qian and Astaneh-Asl is summarized in Sect. 13.2 of this chapter.

13.1 Coupled Bays and CFT Columns

This system was developed by Skilling Ward Magnusson Barkshire (now Magnusson Klemencic Associates) and was tested and studied at the University of California Berkeley by the first author and his research Associates (Zhao and Astaneh-Asl 2008, 2004b). Figure 22 shows components of the system. Specimen One, which has a height to width ratio of 1.5 for the infill plate, yielded at the inter-story drift of 0.006 and showed very high cyclic ductility and was able to reach cyclic inter-story drift of about ±3.3% after undergoing 79 cycles, 35 of the cycles being inelastic. The maximum shear force reached was about 4079 kN (917 kips) during the 79th cycle. Throughout the test, the gravity load carrying concrete-filled steel tube remained essentially elastic while non-gravity carrying lateral load resisting elements underwent well-distributed and desirable yielding. The specimen failed during the 80th cycle due to a fracture of the top coupling beam due to low-cycle fatigue fracture of the web. Figure 23 shows Specimen One before testing and its cyclic behavior.

Similar to Specimen One, Specimen Two also behaved in a ductile and desirable manner. Up to inter-story drifts of about 0.7%, the specimen was almost elastic. At this drift level, some yield lines appeared on the wall plate, and the force-displacement curve started to deviate from the straight elastic line. During later cycles, a distinct X-shaped yield line was visible on the steel plate shear wall. The specimen could tolerate 79 cycles, out of which 30 cycles were inelastic. The specimen reached an inter-story drift of more than 2.2% and maximum shear force of 5449 kN (1,225 kips). At this level of drift, the upper floor-coupling beam fractured at the face of the column (due to low-cycle fatigue), and the shear strength of the specimen dropped to about 88% of the maximum shear force reached in previous cycles (5449 kN or 1,225 kips). Testing continued, and when inter-story drift reached about 3.2%, the load dropped to below 75% of maximum load and test stopped.

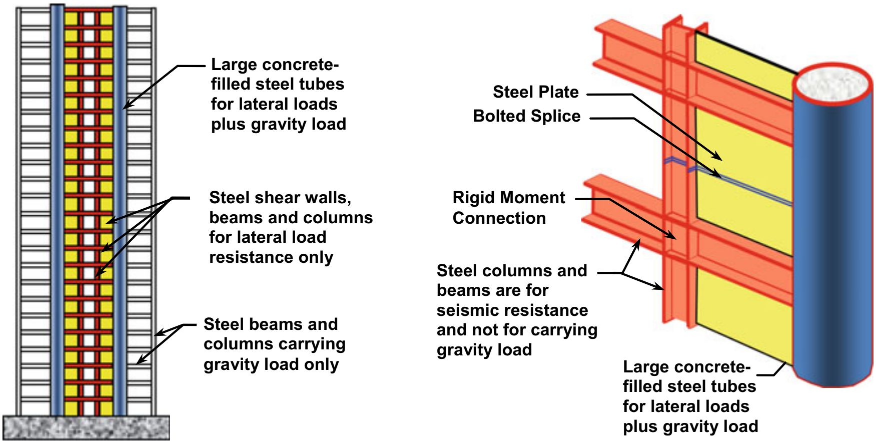


Fig. 22 Components of innovative coupled wall with CFT columns

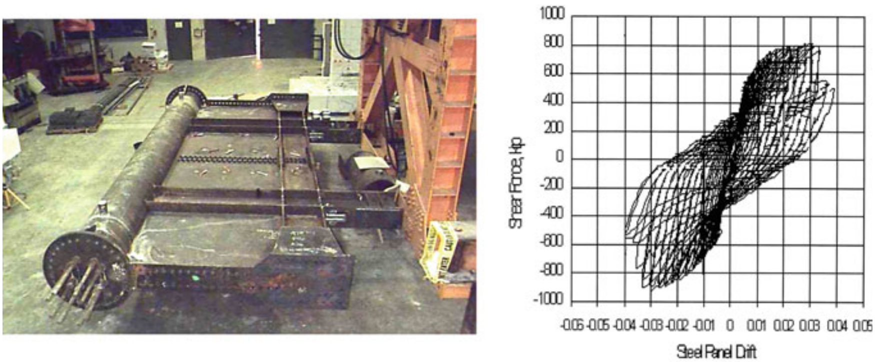


Fig. 23 Specimen one, and its shear force-drift relation

13.2 The High-Performance Steel Plate Shear Wall (Qian and Astaneh-Asl 2017)

Figure 24 shows the main elements of the new *High-Performance Steel Plate Shear Wall* (*HPSPSW*) system, as well as its main advantages compared to the current steel plate shear walls in the current design codes such as the AISC-341 standard (AISC 2016a).

The main elements of the new HPSPSW are:

a. **Unstiffened steel plate shear wall**: the unstiffened steel plate is designed to satisfy two levels of performance: (1) to resist the story shear under the ultimate factored load primarily by the strength of the tension field action; and (2) to remain essentially elastic under the service wind and service level lateral seismic code design forces. The main innovation in this system is that the steel infill plate is not connected to the boundary columns; instead, the infill plate is connected to a vertical stiffener, usually a T-section next to the column. Separating the steel infill plate from the column in this system frees the columns from the lateral forces of the tension field action reducing bending moment in the columns significantly.

b. **Vertical side stiffeners**: As shown in Fig. 24, two vertical steel plates, T-sections, or another steel shape, are shop-welded to the vertical sides of the infill steel plate and are next to but not connected to the columns. These side stiffeners are located between floors and play three important roles: (1) to prevent lateral forces of the tension field action acting on the columns, (2) to provide out-of-plane buckling restraint to the steel infill plate, and (3) to provide in-plane boundary constraint to the tension field action;
c. **Beam-to-column moment connections**: The beam-to-column connections in the new *HPSPSW* system presented here are the new innovative Gusset Plate Moment Connection (GPMC), which is also developed by the authors (Qian and Astaneh-Asl 2016b, 2017) and discussed in the previous section.
d. **Boundary columns**: The boundary columns in the *HPSPSW* are not connected to the steel shear wall between the floor beams, preventing the steel plate from applying lateral force to the columns. Since columns are not connected to the infill plate directly, the columns can be steel rolled or built-up open or closed sections, steel-concrete composite sections, or even reinforced concrete sections. There is no restriction on the column orientation. The role of the column in this system is similar to the role of the columns in concentrically braced frames—to carry primarily axial loads; and
e. **Boundary beams**: in the new *HPSPSW* system, the boundary beams can be fillet-welded or bolted to the steel infill panel. If welds are used, welding can

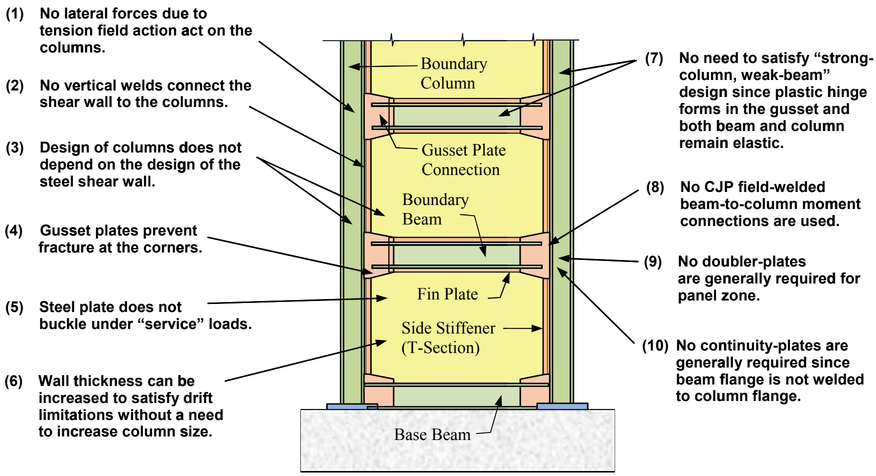


Fig. 24 The new HPSPSW with its advantages

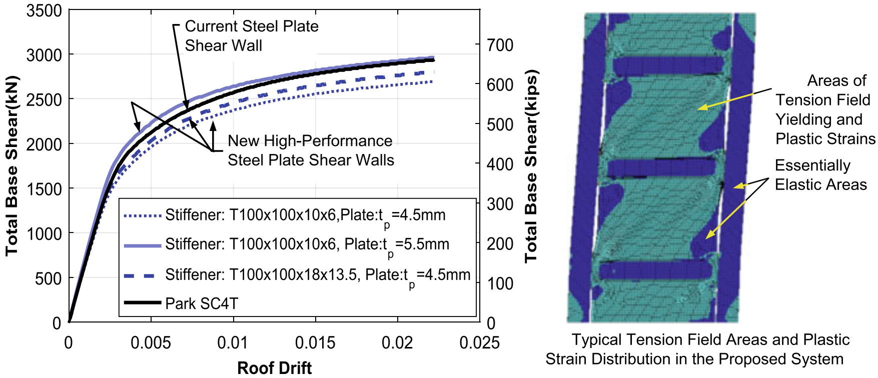


Fig. 25 Pushover curve and von Mises stresses in the innovative, high-performance steel plate shear wall

be done in the field or the shop in the "modular" construction option of the new system. These welds are fillet welds, requiring only visual inspections and not ultrasonic testing. The beams in the new system do not need to be wide flange shape as is the case in the current SPSW system; they can be angles, channels, or even simple flanges as long as they are strong enough to carry the gravity loads and the tension field forces from the wall.

Figure 25 shows a comparison of pushover curves in a Steel Plate Shear Wall system designed according to the AISC-341 standard (AISC 2016a) and same system designed as a High-Performance Steel Plate Shear Wall as shown in Fig. 24. As also shown in Fig. 25, the columns and beams in the HPSPSW remain essentially elastic, very small bending moments are generated in the columns, and the bulk of yielding takes place in the steel infill plate and some yielding in the connection gusset plate as per intended design. More information on the behavior and design of HPSPSW is in Qian and Astaneh-Asl (2017). To compare the HPSPSW to current SPSW system included in the AISC-341 standard AISC-341 standard (AISC 2016a) using the 9-story SPSW building in the AISC Design Guide 20 (AISC 2007) the SPSW shear walls were redesigned as HPSPSW system to resist the same lateral forces with the same inter-story drift limitations. It was shown that the HPSPSW designs could be 20% (code-based designed) to 10% (performance-based design) lighter than the AISC SPSW design in the AISC Design Guide 20 (AISC 2007), not including the additional savings on fabrication. The HPSPSW system had much lighter column sections compared to the AISC SPSW design. The comparison between the performance of the new system and the current AISC steel plate shear wall system (Qian and Astaneh-Asl 2016a, b, 2017) showed that the new system reduces the structural weight and column over-stress significantly, eases the design iterations and improves the constructability, material-utilization ratio, and economy of the system efficiently with enhanced seismic performance.

The High-Performance Steel Plate Shear Wall system has high potential to be an efficient, economical and easy to design, construct and inspect the system. There is a need for actual cyclic tests of the system to establish its performance under cyclic loading.

13.3 Buckling Restrained Steel Shear Walls

Another solution to reduce the tension field forces acting on the boundary columns has been to add buckling-restraining braces to the steel infill plate to delay or prevent its buckling before its yielding in shear (Nie et al. 2012; Tsai et al. 2007, 2010; Li et al. 2010). The major forms of existing buckling restrainers published in the literature include

a. channels welded to the infill plate but with a gap left between the restrainer and the boundary beam (Nie et al. 2012; Lee et al. 2010),
b. square tubes bolted on both sides of the infill plate using enlarged bolt holes to allow the relative movement of the infill plate and the buckling restrainers (Li et al. 2010; Tsai et al. 2007, 2010),
c. concrete cover plates added to the steel shear wall infill plate, where there is a gap around the concrete wall to prevent engagement of concrete with the boundary frames during small and moderate, but more frequent earthquakes (Zhao and Astaneh-Asl 2004a; Astaneh-Asl 2002a) The concrete plate engages with the steel shear wall making the wall a composite shear wall during major seismic events.
d. An infill steel panel sandwiched between two concrete cover plates similar to that in a composite shear wall but with enlarged bolt holes so that the concrete cover could slide freely relative to the steel plate (Guo et al. 2009).

13.4 Steel Slit Panel-Frame Shear Walls

Hitaka and Matsui (2003) were the first to introduce a series of slits in the unstiffened steel shear wall. The use of a series of slits on the infill panel enables the formation of a series of flexural links in between the slits and allows the system to provide a fairly ductile response without the need of substantial stiffening (Hitaka and Matsui 2003). Cortes and Liu (2011a, b) further improved this concept and studied steel slit panel—the frame, a new steel shear wall system, and developed design procedures for the system. The main components of the systems are columns and beams and Steel Slit Panels (SSPs). The beam-to-column connections are simple shear connections. The panels are not connected to the column, thus preventing the development and application of the tension field action forces to the columns. The Steel Slit Panel has some vertical slit cuts. The slit cuts convert the steel infill plate into a series of vertical beam. The story shear is resisted by vertical beams bending in double curvature and forming plastic hinges at the top and bottom ends of the vertical beams. The panels have stiffeners in their vertical edges to prevent out of plane buckling of the panels.

Based on their analytical and actual cyclic testing of scaled specimens, the researchers concluded that all specimens were able to undergo cyclic inter-story drifts of at least 5% without the peak strength at each cycle dropping below 80% of the maximum strength reached during the first cycle. The proposed analytical equation could predict the stiffness and strength of the SSPF. The results also indicated that the stiffness is significantly reduced by the characteristics and properties of the boundary frames, especially by the flexural rigidity of the boundary beams. The researchers state that “The experimental results suggest that the SSPF system has high potential, with ductile and predictable behavior. Further studies confirming the performance of this system must be conducted before the SSPF may be used for lateral resistance in seismic regions.”

13.5 Perforated Steel Shear Walls

Vian et al. (2009) tested steel perforated shear walls, where multiple circular holes were cut out of the infill plate. The holes were arranged in regularly spaced diagonal strips within the infill panel. One of the objectives of cutting out the circular holes was to reduce the strength and stiffness of the infill panel. The researchers stated that: “This option may be beneficial to designers who feel an SPSW system is suited to a particular structure, but the required solid infill plate thickness is unreasonably small.” The steel used in the perforated panel was low yield steel, and the beam-to-column connections were special “reduced beam section” moment connections. The tested specimens behaved in a ductile manner and resisted cyclic lateral story drifts of 3% or greater. The researchers state that the perforation holes reduce the capacity of the tension field and to achieve the same shear strength for the infill panel the thickness is increased. Ignoring the additional cost of cutting multiple holes in the infill plate, the concept does not reduce significantly, the lateral forces acting on boundary elements, especially columns, which is the main reason for making the traditional steel shear walls not as cost-efficient as they can be.

14 Summary

Since 1960s steel shear walls are used as an efficient lateral force resisting system. Steel shear walls consist of a steel infill plate connected to boundary columns and beams by welded or bolted connections. The connection of the boundary columns and beams are usually moment connections making the boundary frame a moment frame. The steel plate is designed to resist the story shear, while the entire system consisting of the steel infill plate and boundary frame resists overturning moments.

Steel infill plate can have only horizontal or vertical stiffeners or both. Without stiffeners, due to relatively large height-to-thickness ratio, the unstiffened steel infill plate buckles diagonally under relatively small shear and after buckling resists the applied shear by diagonal tension field action. Unstiffened steel shear walls are popular in North America with a 55-story building in Los Angeles currently being the tallest steel shear wall building in the world.

Two issues make the unstiffened steel shear walls relatively expensive compared to the stiffened shear walls. One is that buckling of steel infill plate can occur under service loads which is currently an unacceptable performance in some countries like China. By adding stiffeners to the steel infill plates, we can delay the diagonal buckling of the steel infill plate under service load, and even avoid the buckling until the infill plate yields in shear during ultimate design loads. The second issue with unstiffened steel shear walls is that the development of tension field action after buckling of the compression diagonals results in the application of relatively large lateral tension field action forces to the columns making the columns relatively heavy. Stiffened shear walls are popular in Asia, and currently, a 73-story high-rise building in China is the tallest steel shear wall building in the world. The building has stiffened shear walls designed to prevent diagonal buckling of the wall under more frequent earthquakes, but allow buckling of compression diagonal and development of tension field action during severe earthquakes. This dual design criterion appears to be more rational than the current approach in the North American design codes that only considers designing the steel infill panel to resist the shear forces expected during the strong earthquakes without any consideration given to the performance of the wall during small but more frequent earthquakes and wind loads.

There is a great potential for new research and development activities in this area to develop more efficient steel shear wall systems as well as more rational seismic design procedures and modeling technics. Several new steel shear wall systems have been developed in recent years primarily to address the main issue with the current unstiffened steel shear wall systems which is the buckling of the diagonal compression field and development of the tension field action, which in turn results in the application of relatively large lateral forces to the boundary columns. Two examples of such systems are the High-Performance Steel Shear Wall System (HPSSWS) (Qian and Astaneh-Asl 2016a, b, 2017), and the steel slit panel frame shear walls (Cortes and Liu 2011a, b). In the HPSSWS the infill plate is connected to the boundary beams but not to the boundary columns eliminating any lateral force applied to the columns. In this system, the vertical edges of the steel infill plate, instead of being connected to the columns, are connected to the plate or Tee-section stiffeners. In the steel slit panel-frame shear wall systems, the steel infill plate is connected to the boundary beams and columns, but the infill wall has a series of vertical slits cut into it making the plate act as a collection of vertical columns acting primarily in bending.

At this time, steel plate shear walls are one of the most resilient lateral load resisting systems, with sufficient initial stiffness, yield strength, and ductility to resist lateral loads of winds and earthquakes in a cost-efficient manner. Still, there is plenty of room for innovative ideas to make the system more efficient by developing new and cost-efficient concepts and detailings in the following areas:

1. More analytical and especially experimental research is needed on the two promising steel shear wall systems, the High-Performance Steel Plate Shear Walls (Qian and Astaneh-Asl 2016a, b, 2017) and the Slit Panel-Frames (Cortes and Liu 2011a, b), to establish their performance and to develop design procedures for these and other future systems that can eliminate the lateral tension field action on the columns.
2. More analytical and experimental research is needed to establish which of the simple, partially restrained and rigid moment connections are more efficient for the beam-to-column and column-base connections. Then, based on the findings of such research, new or improved cost efficient beam-to-column and column-base connections can be developed.
3. There is a need for current codes such as the AISC-341 standard (AISC 2016a) to have a design procedure that includes satisfactory performance under both frequent but medium size seismic events and less frequent but large earthquakes. This is the case in Chinese code but not in the U.S., Canada, and Eurocode.
4. There is a need to develop details that prevent the transfer of gravity loads to the steel shear walls.
5. There is a need to develop construction procedures that increases pre-fabrication and shop work as much as possible and reduces field work, especially field welding, inspection, as well as erection time.

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