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Review on Steel Plate Shear Wall for Tall Buildings

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Abstract: ***For construction activity normally we use materials as concrete and steel to build up tall buildings. In concrete there are different constituents like aggregate, cement, sand, admixtures, water and plasticizers from which we can achieve the characteristic strength according to our structure. We also use various grades of steel like MS, TOR, TMT, depending on the type of structure. We can construct building by using these two main components up to the limit that means the deign limits according specified by the IS 456:2000 'Plain and Reinforced Concrete'. But for the high rise structures we cannot go only by using these two components i.e. concrete and steel. We have to choose some different alternatives or different systems to construct the high rising structures therefore we can see system like Steel plate Shear Wall (SPSW) suggested by different scientist that we are going to study in this paper. We are going to study the Performance of Steel Plate Shear Wall during Past Earthquakes events. In this paper we will also study the testing on steel plate and also the different case study of SPSW system.***

Keywords: Tall building, Steel plate sheer wall, testing on SPSW, Shear loads, Deflection, Floors

1. Introduction

1.1 General

The main function of steel plate shear wall is to resist horizontal story shear and overturning moment due to lateral loads. Steel plate shear walls (SPSW) can be use as a lateral load resisting system for buildings. A typical SPSW (Fig. 1) consists of stiff horizontal and vertical boundary elements (HBE and VBE) and infill plates. The resulting system is a stiff cantilever wall which resembles a vertical plate girder.

There are two types of SPSW systems, which are the standard system and the dual system. (Fig.2 & 3). In the standard system SPSW is used as the sole lateral load resisting system and pin type beam to column connections are used in the rest of the steel framing. In the latter system, SPSW is a part of a lateral load resisting system and installed in a moment resisting frame. In this case forces are resisted by the frame and SPSW. SPSW can have stiffened or unstiffened infill plates depending on the design philosophy.

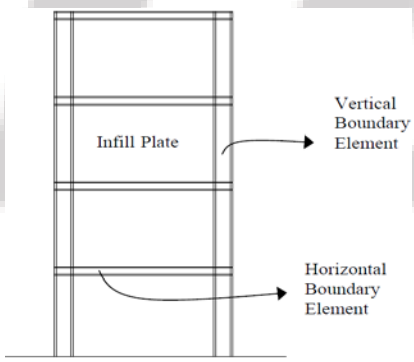


Figure 1: Typical Steel Plate Shear Wall

Earlier designs used stiffeners to prevent buckling of infill plates under shear stresses. On the other hand, more recent approaches rely on post buckling strength. Based on the work of Wagner, it has been known that buckling does not necessarily represent the limit of structural usefulness and there is considerable post buckling strength possessed by restrained unstiffened thin plates. At the onset of buckling, this occurs at very low lateral loads, the load carrying mechanism changes from in-plane shear to an inclined tension field. The additional post buckling strength due to the formation of tension field can be utilized to resist lateral forces. Due to the cost associated with stiffeners most new designs employ unstiffened infill plates.

Design recommendations for SPSW systems are newly introduced into the AISC Seismic Provisions for Structural Steel Building. These provisions basically present guidelines on the calculation of lateral load capacity of SPSW as well as recommendations on the seismic characteristics. Lateral load resisting capacity of SPSW systems has been studied experimentally and numerically in the past and procedures for computing the nominal capacity are developed. These experimental and analytical studies led to the development of code provisions. [1].

The high rise buildings mostly fail due to bucking therefore we have use to SPSW system for the lateral force resisting system. In this paper we will study the behaviour of SPSW. [5]

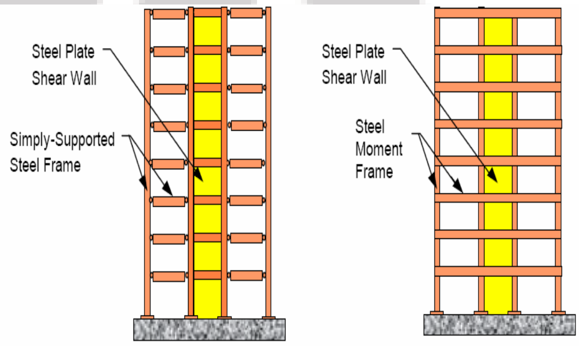


Figure 2: (a) Standard SPSW system (b) Dual SPSW system

1.2 Advantages of SPSW

1. Steel shear walls are very efficient and economical lateral load resisting systems.
2. The steel shear wall system has relatively high initial stiffness, thus very effective in limiting the drift.
3. Compared to reinforced concrete shear walls, the steel shear wall is much lighter which can result in less weight to be carried by the columns and foundations as well as less seismic load due to reduced mass of the structure.
4. By using shop-welded, field-bolted steel shear walls, one can speed-up the erection process and reduce the cost of construction, field inspection and quality control resulting in making these systems even more efficient.
5. Due to relatively small thickness of steel plate shear walls compared to reinforced concrete shear walls, from architectural point of view, steel plate shear walls occupy much less space than the equivalent reinforced concrete shear walls. In high-rises, if reinforced concrete shear walls are used, the walls in lower floors become very thick and occupy large area of the floor plan.
6. Compared to reinforced concrete shear walls, steel plate shear walls can be much easier and faster to construct when they are used in seismic retrofit of existing building.
7. Steel plate shear wall systems that can be constructed with shop welded-field bolted elements can make the steel plate shear walls more efficient than the traditional systems. [1].

2. Literature Review on Performance of Steel Plate Shear Wall During Past Earthquakes

Since 1970's, steel shear walls have been used as the primary lateral load resisting system in several modern and important structures. Initially, and during 1970's, stiffened steel shear walls were used in Japan in new construction and in the U.S. for seismic retrofit of the existing buildings as well as in new buildings. In 1980's and 90's, un-stiffened steel plate shear walls were used in buildings in the United States and Canada. In some cases, the steel plate shear walls were covered with concrete forming a somewhat composite shear wall. In the following a brief summary of the applications of steel plate shear walls, stiffened or un-stiffened is provided.

According to Thorburn (1983) it is believed that this building, referred to as Nippon Steel Building, was the first major building using steel plate shear walls. Located in Tokyo, it was completed in 1970.

The lateral load resisting system in longitudinal direction was a combination of moment frame and steel plate shear wall units in an H configuration and in transverse direction consisted of steel plate shear walls. The steel plate wall panels consisted of 9' by 12'-2" steel plates with horizontal and vertical steel channel stiffeners. The thickness of steel wall plates ranged from 3/16" to ½ ". In design, the gravity load was not given to steel shear walls and the walls were designed to resist design lateral loads without buckling.

The structure was initially designed using reinforced concrete shear walls. However, according to Engineering News Record (1978), due to patent problem, the R/C walls were converted to steel shear walls. According to ENR article (ENR, 1978), "the contractor rejected a steel braced building core as too expensive" compared to steel shear wall.

The structure consisted of moment perimeter frame and "T" shaped stiffened steel shear walls. The wall panels were about 10-ft high and 16.5 feet long and had vertical stiffeners on one side and horizontal stiffener on the other side. The panels were connected to boundary box and H steel columns using bolts. The construction contractor in this case has made a comment that "The next high-rise building we do won't likely be designed with bolted steel seismic walls" (ENR, 1978). According to ENR article, the contractor on another high-rise in Tokyo switched from bolted steel panels to welded panels after failing to achieve the required precision.

This structure, described in Reference (Troy and Richard, 1988) is a very good example of efficient use of steel shear walls in areas with low seismicity but with relatively high wind loads. The 30-story structure has steel braced frame in longitudinal direction and steel plate shear walls in the transverse direction. The shear walls in this structure carry about 60% of the tributary gravity load while the wide flange columns at the boundary of shear walls resist the remaining 40%.

By using steel plate shear walls as gravity load carrying elements, the designers have saved a significant amount of steel in beams and columns and compared to equivalent steel moment resisting frame, the steel shear wall system has used 1/3 less steel (Troy and Richard, 1988). Located in Dallas, the wind loads were the governing lateral loads. Under the design wind load, maximum drift was only 0.0025. The relatively low drift is due to relatively high in-plane stiffness of steel plate shear walls.

This structure is a good example of the use of steel shear walls in an "important" structure such as a hospital in an area of very high seismicity such as California. The hospital building is a replacement for the reinforced concrete Olive View Hospital that had partially collapsed during the 1971 San Fernando earthquake and had to be demolished. In the new Sylmar Hospital, the gravity load is resisted entirely by a steel space frame and the lateral load is resisted by the reinforced concrete shear walls in the first two stories and steel plate shear walls in the upper four stories. The steel shear wall panels in this building are 25 ft wide and 15.5 feet high with thickness of wall plate being 5/8" and 3/4". The walls have window openings in them and stiffeners. The steel plate panels are bolted to the fin plates on the columns. The horizontal beams as well as the stiffeners are double channels welded to the steel plate to form a box shape. According to the designers, the double channel box sections were used to form torsion ally stiff elements at the boundaries of steel plates and to increase buckling capacity of the plate panels.

The walls were designed for global buckling capacity of the stiffened walls as well as local buckling capacity of the panels bounded by the stiffeners. The tension field action capacity was not used although the designers acknowledge its presence and consider the strength of tension field action as a "second line of defense" mechanism in the event of a maximum credible earthquake. The California Strong Motion Instrumentation Program (CSMIP) has instrumented the Sylmar hospital. The 1987 Whittier and the 1994 Northridge earthquakes shook the structure and valuable records on response of the structure were obtained. The accelerations at roof level were more than 2.3g while the ground acceleration was about 0.66g. The investigation of damage to this building in the aftermath of the 1994 Northridge earthquake by the author indicated that there was severe damage to some non-structural elements such as suspended ceilings and sprinkler system resulting in breakage of a number of sprinklers and flooding of some floors. In addition, most TV sets bolted to the wall of the patients' rooms had broken the connections to the wall and were thrown to the floor. The non-structural damage was clearly an indicator of very high stiffness of this structure, which was also the cause of relatively large amplification of accelerations from ground level to roof level. [1]

One of the most important buildings with steel plate shear walls in a very highly seismic area is the 35-story high-rise in Kobe, Japan. The structure was constructed in 1988 and was subjected to the 1995 Kobe earthquake. The structural system in this building consists of a dual system of steel moment frames and shear walls. The shear walls in the three basement levels are reinforced concrete and in the first and second floors the walls are composite walls and above the 2nd floor the walls are stiffened steel shear walls. The author visited this building about two weeks after the 1995 Kobe earthquake and found no visible damage.

Studies of this structure have indicated that the damage was minor and consisted of local buckling of stiffened steel plate shear walls on the 26th story and a permanent roof drift of 225mm in northerly and 35mm in westerly directions. The results of post-earthquake inelastic analyses of this structure reported in above references indicate that soft stories may have formed at floors between 24th and 28th level of the building. The maximum inter-story drift is about 1.7% in 29th floor of the NS frame.

Currently, the tallest building with steel plate shear walls in a very highly seismic area of the United States is a 52-story building in San Francisco. The building is a residential tower and when completed will have 48 stories above ground and four basement parking levels.

The gravity load carrying system in this building consists of four large concrete-filled steel tubes at the core and sixteen concrete-filled smaller steel tube columns in the perimeter. The floors outside the core consist of post-tensioned flat slabs and inside the core and lower floors are typical composite steel deck-concrete slab. The foundation consists of a single reinforced concrete mat foundation.

The main lateral load resisting system of the structure consists of a core made of four large concrete field steel tubes, one at each corner of the core, and steel shear walls and coupling beams. There are built-up H columns between the two corner pipe columns. The steel shear walls are connected to concrete filled steel tubes by coupling beams. The shear wall units are primarily shop-welded and bolt spliced at the site at each floor mid-height. The only field welding is the connection of the girders and steel plate shear wall to the large concrete-filled steel tube columns.

Similar to the 52-story structure discussed in previous section, the steel plate shear wall system in this building also is primarily shop-welded, field bolted with only steel plates and girders welded to the round columns in the field. Four round concrete-filled tubes carry the bulk of gravity in the interior of the building. The I-shaped columns within the steel box core do not participate in carrying gravity and are primarily part of the lateral load resisting system which can be considered to be a dual system of steel shear wall and special moment-resisting frames.[1]

3. Past Research On Steel Plate Shear Wall

A number of researchers in United States, Japan, Canada and United Kingdom have studied behavior of steel shear walls [1]. Researchers at the University of Alberta have conducted monotonic and cyclic tests of un-stiffened steel plate shear walls. The load displacement curve indicates a ductile behavior and significant over-strength.

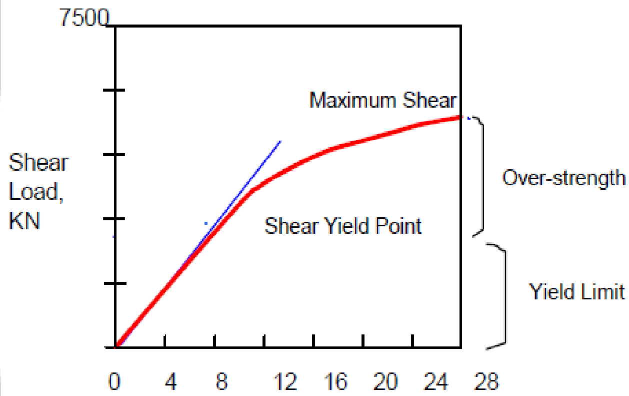


Figure 3: Load Displacement curve

The specimen exhibited a ductility exceeding 4.0. Earlier, Thorburn based on their analytical research, had proposed an equation for angle of inclination of tension field. The test indicated that the proposed equation is sufficiently accurate. The figure 5 also shows load-displacement cyclic hysteresis response. The test results indicate over-strength of more than 2.0 and a ductility of more than 3.5.

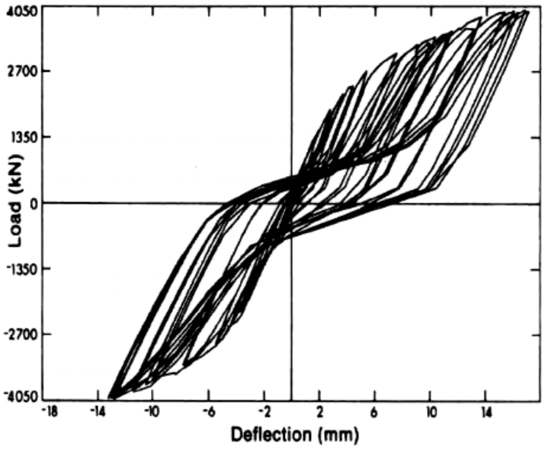
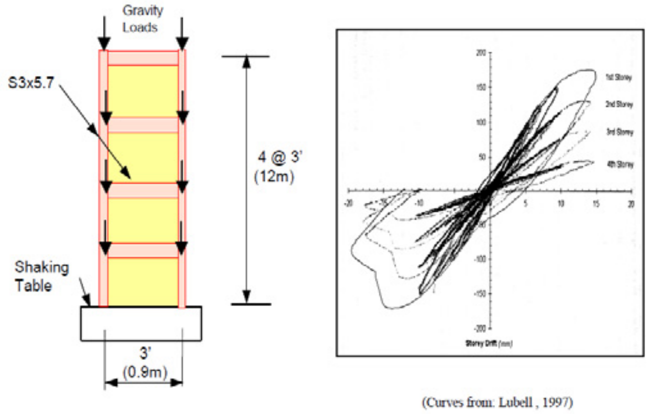


Figure 4: Load –displacement cyclic hysteretic response

The failure mode was fracture of left column at the heat-affected zone of weld connecting the column to the base plate. The researchers related this failure mode to local buckling of column that had occurred during cycle 20 causing large deformation amplitudes at locally buckled areas of the column flange. Prior to fracture, the specimen behaved in a very ductile manner. Unfortunately, failure mode of this specimen was not directly related to shear neither failure of the wall itself nor the behavior of the system as a whole. The failure at the base of the column where it was attached to reaction beam was probably due to stress concentration at the base of the specimen where it was connected to the reaction floor and test set-up. Such stress concentrations are not expected to occur in a real structure. However, even with premature failure of the base of column in this specimen, the cyclic behavior indicates over-strength of about 1.3 and a ductility of more than 6.0.

Recently researchers at the University of British Columbia have completed a series of cyclic and shaking table tests of steel plate shear walls. In these studies, cyclic shear loads were applied to two single story specimens. The boundary frames in the specimens were moment frames resulting in a "dual" structural system. The two specimens differed only in the base gusset plate details and the top beam. For second specimen, stronger base connections and top beam were used. The single story specimens experienced significant inelastic deformations up to ductility of about six. The over strength was about 1.5. The researchers concluded that the two one story specimens demonstrated that the infill steel plates significantly reduced demand on the moment-resisting frame by producing redundant diagonal story braces that alleviated the rotation demand on the beam-to-column connections.



Specimen on Shaking Table   Hysteresis Response

Figure 5: University of Alberta test set-up and a sample hysteric behavior.

In the shaking table tests, a four-story specimen representing 30% scale model of inner core of a residential building was used. Figure 2.5 shows a view of the specimen and the test set-up. The dimensions of each story were almost the same as the one-story specimens. The frame was welded rigid frame making the system a dual system. In each panel of the specimen, a maximum displacement ductility of 1.5 was achieved prior to a global instability failure propagated by yielding of the columns. The specimen exhibited over-strength of about 1.20. The specimen proved to be somewhat more flexible than the one-story specimens were. Figure 2.5 shows force-deformation hysteresis loops for the first floor. [1]

Takanashi et al. (1973) and Mimura and Akiyama (1977) have conducted some of the earliest tests of steel shear walls. Takanashi et al. conducted cyclic tests of 12 one-story and two 2-story specimens. The 12 one-story specimens had about 6'-11" (2.1 m) width and 2'-11" (0.9 m) height. They used steel plates with about 3/32", 1/8" and 3/16" (2.3mm, 3.2mm and 4.5mm) thickness. Compared to typical building dimensions, the specimens could be considered to be ¼ – scale of prototype walls. With the exception of one specimen, all specimens had vertical or vertical and horizontal stiffeners welded on one or both sides of the steel plate. The boundary frames were very stiff pin-connected frames. The specimens were loaded along their diagonals to create almost pure shear in the panels. The behaviour of specimens was very ductile and drift angles in some cases exceeded 0.10 radians. The shear strengths of the specimens were predicted well by Von Mises yield criterion given for pure shear as $V_y = A(F_y/\sqrt{3})$.

The two two-story specimens tested by Takanashi et al (1973) were designed to represent shear walls being designed for the high-rise building. The specimens were full scale. One specimen represented the walls with the openings and one without. The specimen with wall opening had a plate thickness of about ¼" (6mm) while the specimen without opening had a wall thickness of about 3/16 (4.5mm). Once again, the shear yield strength predicted by Von Mises yield criterion was in close agreement with test results. The researchers concluded that the conventional beam theory could be used to calculate stiffness and strength of stiffened shear walls. Yamada (1992) reported the results of cyclic tests of steel and composite shear walls. Two specimens were un-stiffened steel plate shear walls. The specimens had a width of 3'-11" (1.2m) and a height of about 2' (0.6 m). The thickness of wall was either 3/64" (1.2mm) or 3/32" (2.3mm). The boundary frames were rigid steel frames encased in rectangular reinforced concrete sections. The specimens were subjected to monotonic load along their diagonal direction. The failure mode was in the form of fracture of base of boundary rigid frames. The behavior of specimens was quite ductile and tension field formed along the diagonal. Sugii and Yamada (1996) have reported the results of cyclic and monotonic tests on 14 steel plates shear walls. The specimens were 1/10 scale model and two stories in height. The boundary frame was rigid composite frame with steel I-shapes encased inside rectangular reinforced concrete sections. Figure 2.6 shows a typical specimen and hysteresis loops. All specimens showed pinching of hysteresis loops due to buckling of compression field.

Torii et al (1996) have studied the application of "low-yield" steel walls in high-rises. In recent years, there have been significant research and development efforts in Japan to use low yield steel in shear walls to control seismic response. Such efforts have led to design and construction of a number of structures using this system (Yamaguchi et al, 1998). From the published data, it appears that this system is very promising and more research and development in this field is needed. Nakashima et al. (1994 and 1995) have tested and reported on the cyclic behaviour of steel shear wall panels made of "low yield" steel. These properties result in relatively early yielding of this type of steel and its sustained and relatively large energy dissipation capability. Tests of low-yield steel subjected to cyclic loads have indicated very stable hysteresis loops and relatively large energy dissipation capability. Figure shows typical hysteresis behavior of specimens.

The specimens were one-story un-stiffened and stiffened walls bolted at the top and bottom to the set-up and subjected to cyclic shear forces. The panels were about 3'-11" by 3'-11" (1.2mx 1.2m). The thickness of all panels was about 15/64" (6mm). Figure also shows a panel during the test. The results of testing of low yield steel shear walls in Japan are significant development in better use of steel in resisting dynamic lateral loads. The Japanese designers have started using the low yield shear panels in buildings. [1]

In the United Kingdom Sabouri-Ghomi and Roberts (1992) and Roberts (1995) have reported results of 16 tests of steel shear panels diagonally loaded. The specimens in these tests consisted of steel plates placed within a 4-hinged frame and connected to it using bolts. Some panels had perforations, Figure being either 12"x12" or 12"x18". The thickness of steel plate was either 1/32" or 3/64". The cyclic load was applied along the diagonal axis resulting in steel plate being subjected to pure shear. The tests indicated that all panels possessed adequate ductility and sustained four large inelastic cycles.

Typical hysteresis loops presented shows specimens reaching a ductility of more than 7 without any decrease in strength. One of the interesting aspects of this test program was to investigate the effects of perforations in the wall on strength and stiffness. The researchers concluded that the strength and stiffness linearly decreases with the increase in (1-D/d). [1]

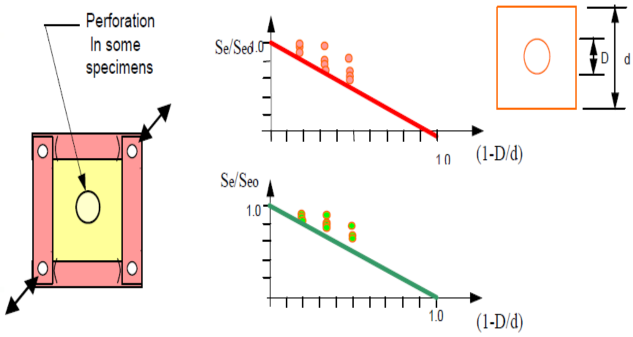


Figure 6: Specimen tested in U.K and the effect of perforation on strength and stiffness

In the United States, research conducted a number of studies of steel plate shear walls. The experimental part of their research included cyclic testing of six, three-story one-bay specimens subjected to cyclic horizontal load at roof level. The specimens were about 1/4 scale and the steel plate shear walls did not have stiffeners. Fig 8 shows the test set-up and the hysteresis loops for these six tests. The studies also included valuable analytical research and resulted in development of analytical models of hysteresis behavior of steel plate shear walls.

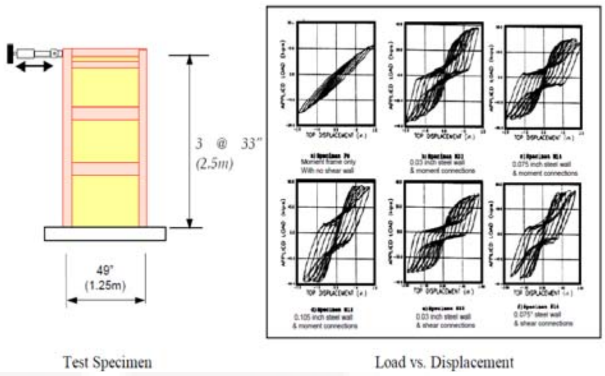


Figure 7: Test set-up and hysteresis behavior of specimens.

Based on the behavior of these six specimens, concluded that when an un-stiffened thin plate is used as shear wall, inelastic behavior commences by yielding of the wall and the strength of the system is governed by plastic hinge formation in the columns. They also concluded that when relatively thick plates are used, the failure mode is governed by column instability and only negligible increase occurs in the strength of the system due to increased thickness of the wall. They suggested " a building can be designed using a thin steel plate shear wall so that it will respond elastically to a minor seismic event or high wind. When subjected to a severe seismic event, walls with less slender plates tend to become unstable due to column buckling before the plate can develop its full strength." In general, the researchers recommended the use of thinner, un-stiffened plates such that the yielding of plate occurs before column buckling. This rational philosophy is incorporated into proposed design recommendations. [1]

4. Conclusion

By the reviewing above papers we can conclude that the steel plate can be used for high rise building to dynamic evaluation of lateral force resisting system. The steel plate shear wall system is depending on the steel what we used and it depends on design specification of building. Also by using SPSW system the stiffness of the building is increased. Then we can adopt this system for multistoried building.

References

[1] A. Astaneh-Asl, "Seismic Behaviour and Design of Steel Shear Walls", SEAONC Seminar, November 2001, San Francisco.
[2] AISC 341-05, 2005. Seismic provisions for structural steel buildings. American Institute for Steel Construction, 2005.
[3] Cem Topkaya, Can Ozan Kurban; " Natural periods of steel plate shear wall system", Middle East Technical University, Turkey. Journal of Constructional Steel Research 65 (2008) 542–551.
[4] K.H. Nip, L.Gardner, A.Y. Elghazouli; "Cyclic testing and numerical modeling of steel tubular bracing members" Engineering Structures 32 (2010) 424-441.
[5] M.B.Jadhav, Patil G.R "Dynamic Evaluation of Lateral Force Resisting Systems for Tall Buildings" Volume III, Issue III, March 2014 IJLTEMAS ISSN 2278 – 2540.
[6] Kyoung Sun Moon;"Sustainable structural systems and configurations for tall building", Yale University, USA. ASCE (2011)

Codes

[1] IS 456:2000; "Plain and Reinforced Concrete Code of Practice" Forth revision.
[2] IS 1893:1984; "Criteria for Earthquake Resistant Design of Structures"

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