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Seismic Behavior and Design of Steel Shear Walls

By

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Introduction

Steel plate shear wall systems have been used in recent years in highly seismic areas to resist lateral loads. Figure 1 shows two basic types of steel shear walls; unstiffened and stiffened with or without openings. Unstiffened shear walls have been very popular in North American applications while in Japan almost all steel shear walls used in recent years have been stiffened.

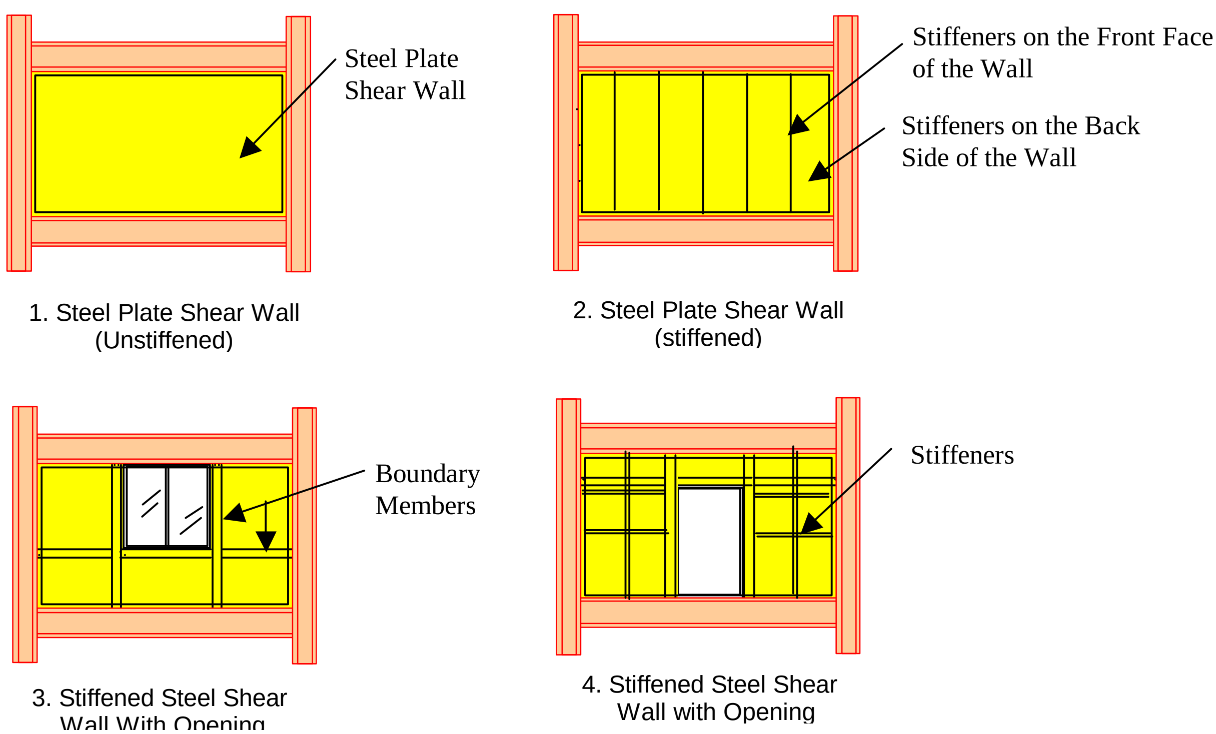


Figure 1. Stiffened and Unstiffened Steel Shear Walls

Some of the advantages of using steel plate shear wall to resist lateral loads are:

1. The system, designed and detailed properly is very ductile and has relatively large energy dissipation capability. As a result, steel shear walls can be 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. These systems can also be very practical and efficient for cold regions where concrete construction may not be economical under very low temperatures.

Since 1970's, in the United States and Japan, a number of important structures using steel plate shear walls have been designed and constructed. A recent Steel Technical Information and Product Report (Steel TIPS Report) by the author (Astaneh-Asl, 2001a) summarizes the information available in the literature on steel shear walls. The Steel TIPS report can be found at www.aisc.org web site and can be downloaded free of charge for personal use. The Steel TIPS report (Astaneh-Asl, 2001a) includes:

1. Introduction to steel shear walls and types of steel shear walls
2. Use of steel shear walls in buildings and seismic performance of such buildings during major earthquakes
3. Results of laboratory tests of steel shear walls
4. Existing and proposed code provisions applicable to seismic design of steel shear walls.
5. Seismic design of steel shear walls
6. Examples of economical and efficient steel shear wall systems

Following sections provide a summary of the above items with added information on further tests done on steel shear walls since publication of the Steel TIPS. In addition, final version of seismic provisions on design of steel shear walls proposed by the author is presented as appendices to this paper and review comments are solicited from the readers. Such comments can be e-mailed to the author by end of 2001 at Astaneh@ce.berkeley.edu.

Applications of Steel Shear Walls and Their Seismic Performance

Since 1970's, a number of steel shear walls have been used in a number of structures in Japan and US. These applications are given in Steel TIPS report (Astaneh-Asl, 2001). Two of these applications have been subjected to relatively large earthquakes and their performance observed. The two buildings are the 6-story Sylmar Hospital in greater Los Angeles area shaken by the 1994 Northridge earthquake and the area and the 35-story building in Kobe, Japan shaken by the 1995 Kobe earthquake.

The 6-story hospital in Los Angeles, California

This structure shown in Figure 3 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.

Figure 3. A view of Sylmar Hospital

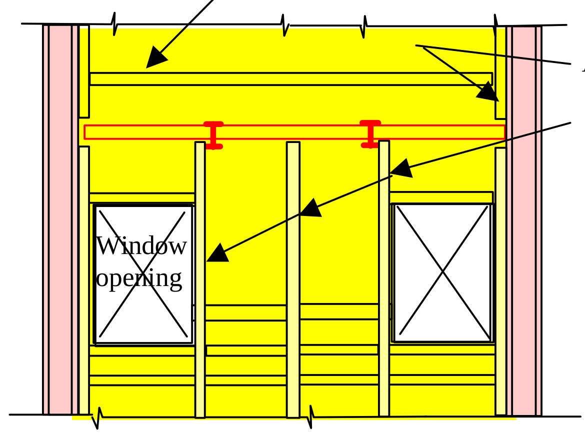


Figure 4. Typical Steel Shear Wall

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 ¾". The walls have window openings in them and stiffeners as shown in Figure 4. 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 as shown in Figure 4. According to the designers, (Youssef, 2000) and (Troy and Richard, 1988) the double channel box sections were used to form torsionally stiff elements at the boundaries of steel plates and to increase buckling capacity of the plate panels.

The California Strong Motion Instrumentation Program (CSMIP) has instrumented the Sylmar hospital. Figure 5 shows data recorded by the CSMIP instruments in this building during the 1994 Northridge earthquake. The acceleration at roof level exceeded 2.3g while the ground acceleration was recorded at 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. More information on seismic response of this structure can be found in (Celebi, 1997).

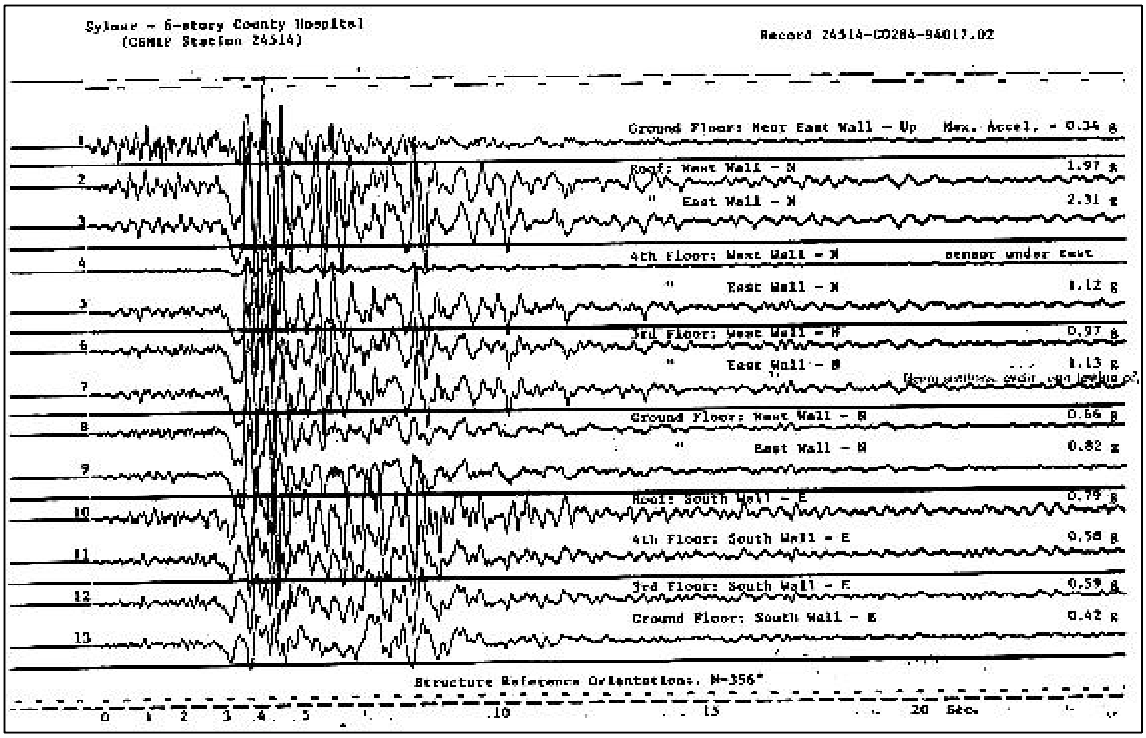


Figure 5. Records obtained from instruments in Sylmar hospital, (CSMIP, 1994)

The 35-story office building in Kobe, Japan

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. Figure 6 shows framing plan and typical frames. The author visited this building about two weeks after the 1995 Kobe earthquake and found no visible damage. 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. Studies of this structure (Fujitani et al., 1996) (AIJ, 1995) 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. Figure 7 shows a view of the building. 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 (AIJ, 1995). The maximum inter-story drift was about 1.7% in 29th floor of the NS frame.

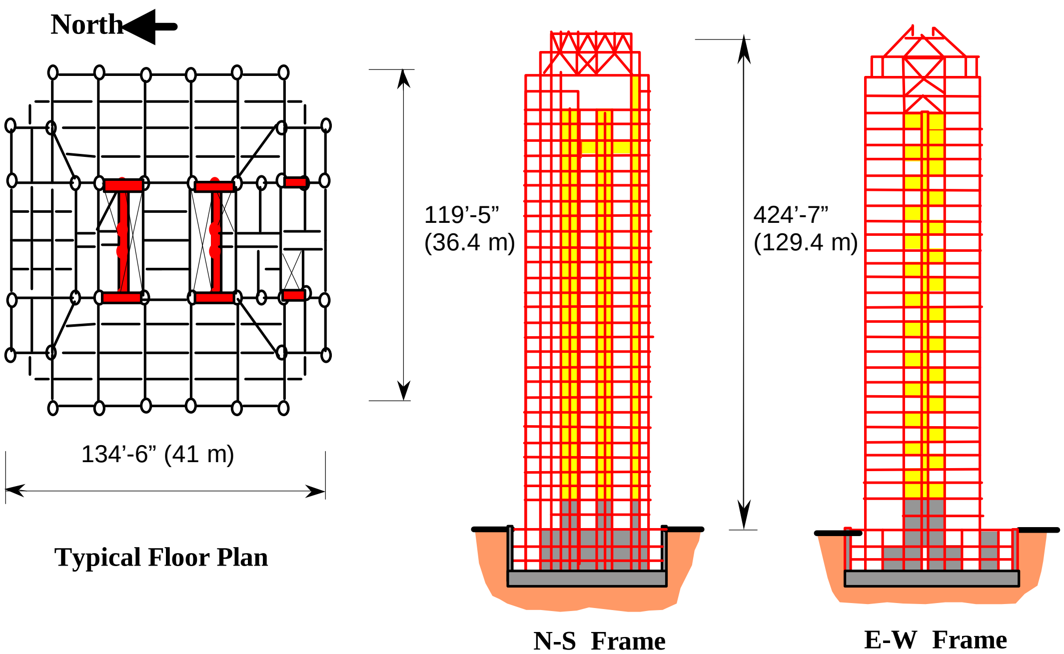


Figure 6. Structure and a view of 35-story Kobe building

(Photo by M. Kanada, from Kanada and Astaneh-Asl, 1996),

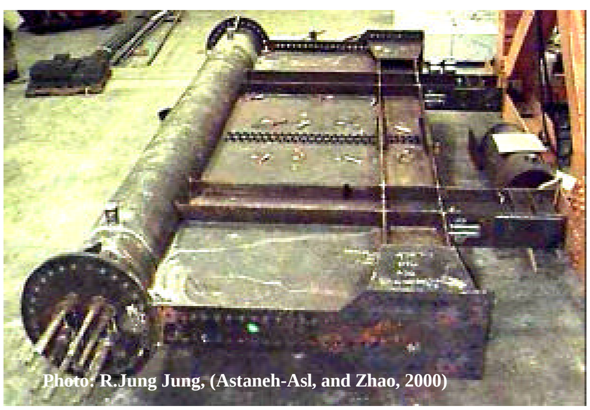
Figure 7. A view of the 35-story Building in Kobe

Tests of Steel Shear Walls in Laboratories

A number of researchers in United States, Japan, Canada and United Kingdom have studied behavior of steel shear walls and have tested their cyclic behavior in laboratories. A more comprehensive summary of these tests is provided in Steel TIPS report (Astaneh-Asl, 2001a) available at www.aisc.org. In the following sections, the tests recently completed by A.Astaneh-Asl and Q. Zhao at the University of California, Berkeley are summarized.

Recently Completed Tests of Steel and Composite Shear Walls at UC-Berkeley

Currently there are two parallel research projects conducted at the Department of Civil and Environmental Engineering of the University of California, Berkeley on shear walls. One is on composite shear walls (Astaneh-Asl and Zhao, 1998-2000) and the other is on steel plate shear walls (Astaneh-Asl and Zhao, 2000-2001). The project on composite shear walls is sponsored by the National Science Foundation. More information on composite shear wall project can be found in (Astaneh-Asl and Zhao, 2001). The information on the behavior and design of composite shear wall will appear in a Steel Tip (Astaneh-Asl, 2001b). In the following, the discussion is limited to the steel plate shear wall tests at UC-Berkeley (Astaneh-Asl and Zhao, 2000).



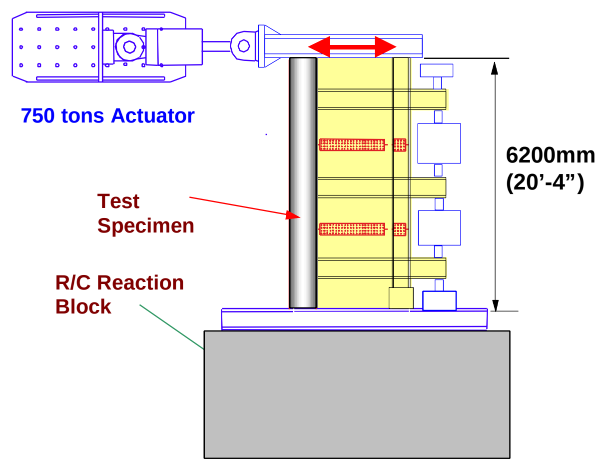


Figure 8. Typical specimen and test set-up (Astaneh-Asl and Zhao, 2000).

Two specimens were tested. The specimens were half-scale realistic representatives of the steel shear wall-moment frame (dual) system used in high-rise structures. Figure 9 shows this steel shear wall system. A number of structures with this type of steel shear wall have been designed by SWMB. The main objectives of the tests were to establish cyclic behavior of steel shear wall systems using concrete filled tubes as boundary elements and internal columns, beams and steel shear walls as the lateral load resisting system. The main parameters studied were stiffness, strength and ductility under cyclic shear displacements. Also, behavior of bolted mid-height splices as well as other connection areas was established. The specimens were realistic ½-scale representative of the actual shear walls used in buildings. The specimens, after instrumentation, were installed in the test set-up, Figure 8, and were subjected to ever-increasing cyclic shear displacements until failure, in the form of large drop of strength, occurred.

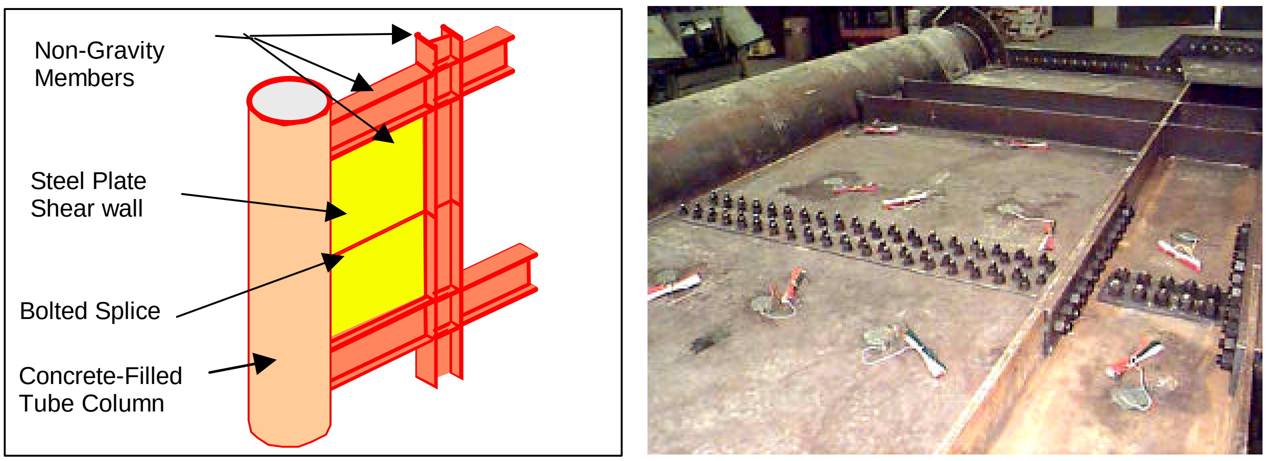


Figure 9. Components of the tested system and bolted splice

Specimen 1 at 3.3% Drift   Specimen 2 at 2.2% Drift

Figure 10. Test Specimens at the End of Test (Astaneh-Asl and Zhao, 2001)

The first specimen, which had shear walls with aspect ratio of 1 (horizontal) to 2 (vertical) and shown in Figure 8, was tested first. The specimen behaved in a very ductile and desirable manner. Up to inter-story drifts of about 0.6%, both specimens were almost elastic. At this drift level some yield lines appeared on the wall plate as well as WF column (non-gravity column). Up to inter-story drifts of about 2.2%, the compression diagonal in the wall panels was buckling and the diagonal tension field was yielding. At this level, in Specimen 1 the WF column developed local buckling. Specimen 1 could tolerate 79 cycles, out which 39 cycles were inelastic, before reaching an inter-story drift of more than 3.3% and maximum shear, strength of about 917 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 60% of the maximum capacity of the specimen.

Specimen 2 behaved in a similar way as Specimen 1 in the sense that they had the same yielding point and therefore same loading history. The yield point for Specimen 2 was at the drift level 0.007. Specimen 2 could tolerate more than 29 cycles, which included 15 inelastic cycles before reaching a drift of 2.2%. At this point while load was about 1240 kips; the top coupling beam fractured and load dropped to about 750 kips.

At the end of each test, the gravity load carrying system was almost intact with almost no damage to the concrete filled tube. The steel plate shear wall had undergone extensive shear yielding over its almost entire area. The I-shape column, a non-gravity carrying column, had also experienced yielding, local buckling at hinge locations and the eventual fracture through locally buckled area. However, none of these events seemed to affect the shear strength of the system. The specimen continued to accept more shear even though the I-shaped column was undergoing deformation and damage. The full results of steel shear wall tests can be found in Astaneh-Asl and Zhao (2000).

Seismic Design of Steel Shear Walls

Shear capacity of steel shear walls can be established using the procedures in the AISC Specification (AISC, 1999) for shear capacity of plate girders. For the background on the equations and why such equations can be used for shear walls, the reader is referred to SSRC Guide (SSRC, 1998) edited by Theodore V. Galambos and Steel TIPS report (Astaneh-Asl, 2001). More detailed procedures and discussion can be found in Steel TIPS report (Astaneh-Asl, 2001), which can be downloaded from www.aisc.org.

Acknowledgements

This paper is based on Steel TIPS report (Astaneh-Asl, 2001a) which was prepared through the support and technical input by the Structural Steel Educational Council (SSEC). The tests reported here were funded by the General Services Administration. The project was conducted in the Department of Civil and Environmental Engineering of the University of California, Berkeley. The Principal Investigator for the project was Professor Abolhassan Astaneh-Asl, Ph.D.; P.E. Graduate student research assistant Qiuhong Zhao is the doctoral student on this project. Staff engineers and machinists from the Department of Civil and Environmental Engineering were part of the research team to assemble the set-up, install the specimen in the set-up, to assist graduate students in instrumentation and in conducting the test and to collect data. The staff included William Mac Cracken, Chris Moy, Jeff Higginbotton Frank Latora, Richard Parson, Mark Troxler and Douglas Zuleikha.

Conclusions and Applications to Seismic Design

Based on results of tests reported here and development of technology summarized in Steel TIPS report (Astaneh-Asl, 2001a) seismic design provisions were developed and proposed by the author. The provisions are in two parts and are attached to this paper as Appendices. Appendix I contains proposed provisions to establish seismic loads for steel shear wall systems. Appendix II contains provisions for seismic design of steel shear walls including provisions on how to establish strength of the wall as well as provisions on detailing to ensure sufficient ductility. The proposed provisions (Appendices I and II) have been proposed in July of 2001 by the author and currently are being reviewed by code writing bodies for modifications and refinement for eventual inclusion in the seismic design codes. The provisions at this time are for information only and anyone using such information takes full responsibility for its use. The reader is encouraged to send her/his comments and questions regarding these provisions to the author at e-mail address: Astaneh@ce.berkeley.edu or fax number (510) 643-5258. Such comments will be greatly appreciated and carefully considered in refining the provisions.

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Appendix I: Proposed Provisions to Establish Earthquake Loads for Steel Plate Shear Wall Systems

The following proposed provisions are for the load side of design equation and intended for possible inclusion in design codes such as the IBC and SEAOC Blue Book.

Values of R-factor, W_o and C_d for steel shear walls:

Design Coefficients and Factors for Steel Shear Walls (Proposed by A. Astaneh-Asl, 2001)

Basic Seismic-force-resisting System	Response Modification Factor, R	System Over-Strength Factor Ω_o	Deflection Amplification Factor, C_d	System Limitations and Building Height Limitations (feet) by Seismic Design Category as Determined in Section 1616.3 of IBC-2000				
				A or B	C	D	E	F
1. Un-stiffened steel plate shear walls inside a gravity carrying steel frame with simple beam to column connections	6.5	2	5	NL	NL	160	160	100
2. Stiffened steel plate shear walls inside a gravity carrying steel frame with simple beam-to-column connections	7.0	2	5	NL	NL	160	160	160
3. Dual system with special steel moment frames and un-stiffened steel plate shear walls	8	2.5	4	NL	NL	NL	NL	NL
4. Dual system with special steel moment frames and stiffened steel plate shear walls	8.5	2.5	4	NL	NL	NL	NL	NL

Note: NL=No Limit

Preferred References to be added to list of References

Astaneh-Asl, A., 2000, "Seismic Behavior and Design of Steel Shear Walls" *Steel Technical Information and Product Services Report, (Steel TIPS)*, Structural Steel Educational Council, Moraga, CA, a copy can be downloaded from: www.aisc.org.

Appendix II: Proposed Provisions for Possible Inclusion in the "AISC Seismic Provisions for Structural Steel Buildings":

16. STEEL SHEAR WALLS (SSW)

16.1. Scope

Steel shear wall systems can be divided into two categories of: (a) "Singular" steel shear wall system where steel shear wall is the only lateral load resisting system and; (b) "Dual" steel shear wall system where steel shear wall is placed parallel to moment frames or within the moment frames and together the steel shear wall and moment frame resist the lateral load. The main elements of a steel shear wall system are the steel shear wall, boundary columns and horizontal floor beams. The steel shear wall itself can be stiffened or un-stiffened.

16.2. Shear Walls

16.2.a. The material of shear wall should be selected such that the $R_y F_y$ of the steel shear wall be less than or equal to the $R_y F_y$ of the boundary columns and horizontal beams connected to the wall.

16.2.b. The design shear strength of steel shear wall shall be established using procedures given in Section G3, Appendix G of the AISC LRFD Specifications for Structural Steel Buildings. Other rational design procedures, based on test results or realistic inelastic analyses can also be used.

16.2.c. At the top floor, if tension field action is used in design, the horizontal beams and boundary columns shall be designed to be strong enough to resist the horizontal and vertical components of the diagonal tension field. Alternatively, alternatively, by using stiffened shear walls or thicker un-stiffened shear walls, the story shear is resisted without utilizing tension field action.

16.2.d. At the bottom floor, where shear wall is attached to the foundation, special arrangements shall be made to ensure proper transfer of horizontal and vertical components of tension field action to the foundation.

16.2.e. In stiffened shear walls, horizontal as well as vertical stiffeners shall be spaced such that the maximum h/t_w of all steel panels bounded by the stiffeners complies with the following:

$$\frac{h}{t_w} \leq 1.1\sqrt{k_v\ E / F_{yw}}$$

The plate buckling coefficient, k_v, is given as:

$$k_v = 5 + \frac{5}{(a/h)^2}$$

Where "a" and "h " are the horizontal and vertical dimensions of the wall panels.

16.2.f. Slip-critical bolts or continuous welds can be used to connect the steel shear wall to the boundary columns and horizontal beams. The connections shall be designed to develop expected shear strength of the wall plate.

16.3. Boundary Columns

16.3.a The web of boundary columns shall be in plane of the steel shear wall. Otherwise, the connection of wall plate to the column web perpendicular to it should be such that out-of-plane bending of column web is prevented.

16.3.b. If boundary columns of steel shear walls are carrying gravity loads, the columns should be designed to remain elastic under the Design Earthquake.

16.3.c. In steel shear wall systems where boundary columns are not carrying gravity load, such columns can be designed to undergo yielding and cyclic local buckling provided that their width thickness ratios be limited to values given in Table I-9.1**.**

16.3.d. The web thickness of boundary columns should be greater than the thickness of the steel plate walls connected to them.

16.3.e. Base connections of the boundary columns to the foundations shall be designed to develop tension yield capacity of the boundary columns. The governing failure mode of a boundary column base connection shall be a ductile failure mode such as yielding of base plate or limited yielding of anchor bolts but not a fracture mode.

16.4. Horizontal Beams

16.4.a. Horizontal beams in a steel shear wall system shall be designed to carry the gravity loads without participation of the steel shear wall.

16.4.b. Web thickness of the horizontal beam shall be greater than the thickness of the steel plate walls above and below the beam.

16.4.c. The shear connection of horizontal beams to boundary columns should be designed to develop shear strength of the beam web. Yielding of the shear plate shall be the governing failure mode of the connection.

16.4.d. In steel shear wall systems where horizontal beams are not carrying gravity load, they can be permitted to undergo yielding and local buckling. Their width-thickness ratios should satisfy limits given in Table I-9.1.

16.5. Dual Shear Wall Systems

16.5.a In Dual shear wall systems where special moment frame(s) are used parallel to the steel shear wall or in the same plane as steel shear wall, the design of special moment frame shall comply with the provisions of Section 9 of this specifications.

16.5.b. In Dual systems, it is preferred that the steel shear wall be an infill to the special moment frame instead of being outside the moment frame and parallel to it.

16.6. Coupling Beams

16.6.a. Steel shear walls can be connected to each other to act as a coupled shear wall system.

16.6.b. Coupling beams shall be connected to the boundary columns with special moment connections designed in compliance with the applicable provisions of Section 9.

16.6.c. Coupling beams shall be compact sections satisfying the width-thickness ratios of Table I-9.1

Preferred References to be added to list of References

Astaneh-Asl, A., 2000, "Seismic Behavior and Design of Steel Shear Walls" *Steel Technical Information and Product Services Report,* Structural Steel Educational Council, Moraga, CA, a copy can be downloaded from: www.aisc.org.

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Proposed Commentary:

C-16. STEEL SHEAR WALLS (SSW)

Cyclic tests of steel shear walls as well as studies of actual behavior of buildings with steel shear walls subjected to major earthquakes have indicated that steel shear walls possess significant ductility and are expected to withstand Design Earthquake by yielding of steel shear wall (Astaneh-Asl, 2001), Astaneh-Asl and Zhao, 2001), (Caccese and Elgaaly, 1993), (Driver et al., 1998), (Rezai et al, 2000). Using the available information, the provisions of this section are formulated.

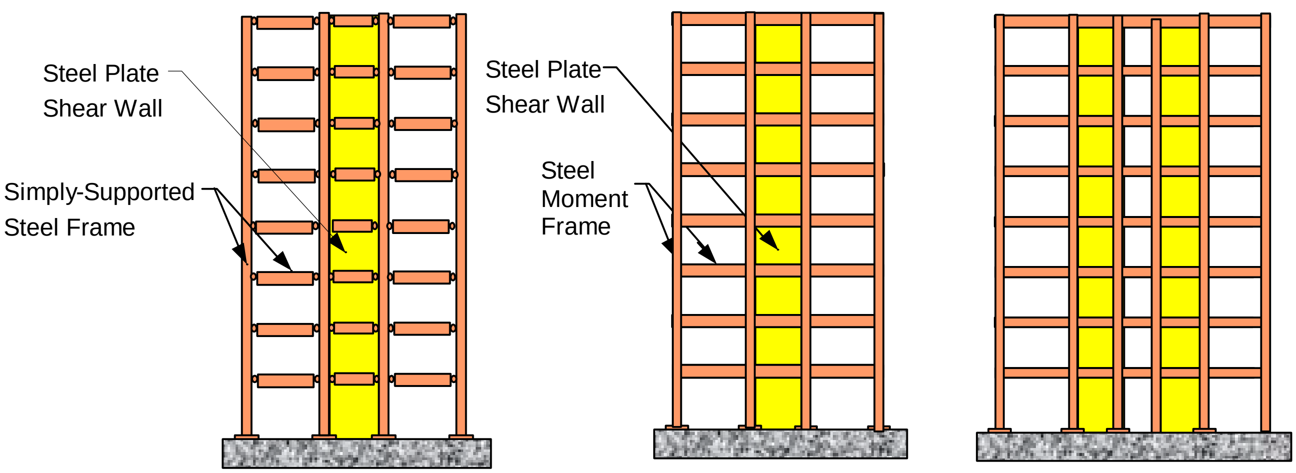
C-16.1. Scope

Steel shear walls covered in these provisions are shown in Figure C-16.1 and are:

(a) “Singular” shear wall system where a steel shear wall is placed inside gravity frame and shear wall is the only element resisting story shear.

(b) “Dual” shear wall system where steel shear wall is placed either inside a special moment frame or is parallel to it. In this Dual system, stel shear wall is designed to resist 100% of the Design Earthquake and special moment frame is designed to resist at least 25% of the Design Earthquake.

(c) Coupled Shear wall system where a coupling beam connects two shear wall bays. The frame or portion of it that contains the shear walls and coupling beams is special moment frame.



a. Singular Shear Wall Inside Gravity Frame

b. Dual System with Shear Wall Inside Moment Frame

c. Dual System with Coupling Beams

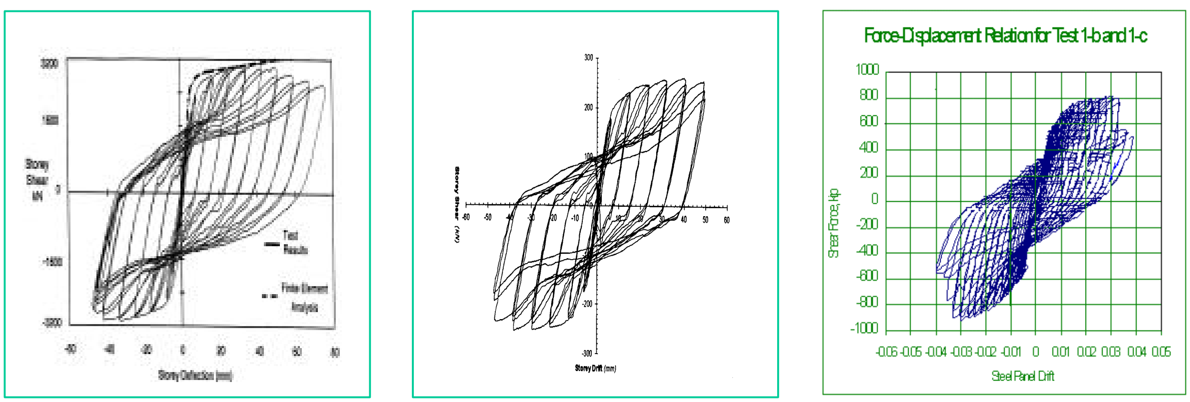
Fig. C-16.1. Typical Steel Shear Wall Systems

C-16.2. Steel Shear Wall

C-16.2.a. Almost all cyclic tests on steel shear walls are done on specimens where the material of the wall had lower than or equal yield point compared to the material of the boundary columns and beams. The result has been that the bulk of yielding, energy dissipation and damage in the system have occurred in the shear wall itself and not in the beams and columns that are quite often responsible to carry gravity loads. To incorporate this desirable behavior into design, and until more test data becomes available on cyclic behavior of shear wall systems where shear wall has higher yield point than the boundary elements, the following provision is recommended:

$$(R_y\, F_y)_{\text{Steel Shear Wall}} \leq (R_y\, F_y)_{\text{Beams and Columns}}$$

The available tests show significant ductility and energy dissipation capacity for steel shear walls. Samples of cyclic behavior of steel plate shear walls are shown in Figure C-16.2. The specimens were capable of tolerating large number of inelastic cycles of shear applications reaching relatively large drift values as shown in Figure C-16-2.



(Driver et al., 1998) (Rezai, Ventura and Prion, 2000) (Astaneh-Asl and Zhao, 2001)

Figure C-16.2. Shear Force- Drift Behavior of Steel Shear Wall Specimens

16.2.b. Unstiffened steel shear walls act primarily as plate girders with steel plate being the web, boundary columns being the flanges and girders being the stiffeners. Since application of AISC procedures to design of plate girders have resulted in economical plate girders with decades of satisfactory performance, application of such equations to design of steel shear walls is recommended. Other procedures such as replacing shear walls with X-braces as done in Japan or replacing shear walls with a series of inclined braces as done in Canada or any other rational method based on actual behavior established by tests can also be used in design.

16.2.c. Similar to last panel of a plate girder, where the web is discontinued, in multi-story shear walls, the beam at the roof level should be designed strong enough to provide anchorage for the tension field action.
16.2.d. At the bottom floor, similar to the roof, either the foundation or a beam placed on or within the foundation should provide anchorage for the tension field action.

16.2.e. Usually, the purpose of providing stiffeners in a shear wall is to delay or to prevent buckling of shear wall plate before it yields. The limitation of $h / t_w \le 1.1\sqrt{k_v\ E / F_{yw}}$ is to prevent buckling of wall panel prior to shear yielding.

16.2.f. Since there is not any cyclic test results on specimens using snug tight bolts or bolts designed for bearing strength but tightened, it is suggested that at this time only slip-critical bolts or welds be used to connect the wall plate to the boundary elements.

16.3. Boundary Columns

16.3.a The main reason for this provision is that the specimens of shear wall tested so far had the web of column in plane of the shear wall.

16.3.b. To design boundary columns that carry gravity load to remain elastic is to provide stability for the building, to prevent lateral creeping collapse, to facilitate return of the frame to its plumb position and most importantly to have undamaged columns to carry the gravity load after the earthquake.

16.3.c. When boundary columns are not carrying gravity and are only to carry seismic loads, such columns can be treated as the shear wall itself and be permitted to undergo yielding.

16.3.d. The main reason for web of column to be made at least as thick as the wall plate is to avoid local yielding in the web of column prior to yielding of shear wall plate.

16.3.e. This provision is to ensure that column base connections in this system are stronger than the members and yielding will be mostly concentrated in the member itself.

16.4. Horizontal Beams

16.4.a. This provision is to prevent significant damage or collapse of the floors after a major earthquake when the steel wall can be permanently buckled. Also, the provision prevents gravity load from being transferred to steel plate shear wall which can cause its buckling if relatively slender wall plate is used.

16.4.b. Same as in boundary columns, the web thickness of the horizontal beam should be designed to be thicker or at least as thick as the wall. In case of beams, the web of the beam is in fact continuation of the walls below and above the beam therefore should not be thinner than the walls.

16.4.c. The aim of this provision is to ensure that the connections of the wall plate to boundary elements remain almost elastic while the wall itself undergoes buckling and yielding. The exception can be properly designed semi-rigid (PR) connections that by yielding and friction slipping can provide extra ductility and energy dissipation capacity for the wall and prevent its excessive yielding.

16.4.d. Similar to non-gravity columns, in a shear wall system if horizontal beams are not carrying gravity, they can be permitted to yield and dissipate energy.

16.5. Dual Shear Wall Systems

16.5.a The information available at this time on actual behavior of dual steel shear wall systems is on dual systems where the moment frames have been *special* frames. This provision is formulated to limit steel shear wall dual system to those with special moment frames.

16.5.b. In Dual systems, it is preferred that the steel shear wall be placed inside the special moment frame. In such systems, the corners of shear wall plate acts as gusset plates above and below the moment connection and results in much less rotation demand placed on such connections. In addition, there is very limited information on cyclic performance of dual shear wall systems where shear wall is placed inside a frame with simple connections but is parallel to a special moment frame. The issues related to transfer of shear from shear walls to moment frames through the floor diaphragms are also not well understood at this time.

16.6. Coupling Beams

16.6.a. Quite often, in order to provide openings, steel shear walls are divided into two or more walls with coupling beams connecting them to each other. Such a system not only can be architecturally desirable but it has been shown by Astaneh-Asl and Zhao, (2001) that such a coupled system is very ductile and desirable from structural point of view.

16.6.b. Obviously if coupling beams are to participate fully in moment frame action, their connections should be special moment connections and designed in compliance with the applicable provisions of Section 9.

16.6.c. This provision is to ensure that the coupling beams are compact enough to participate in inelastic behavior as fully as other members of the system.