Astaneh-Asl, A., (2001), "Seismic Behavior and Design of Steel Shear Walls-SEONC Seminar", Paper Distributed and presented at the 2001 SEOANC Seminar, Structural Engineers Assoc. of Northern California, November 7, 2001, San Francisco.

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	Resp-onse Modifi-cation Factor, R	System Over-Strength Factor Ω_o	Deflection Amplifi-cation 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 **s**teel 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:

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

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.

Astaneh-Asl, A. and Zhao, Q., (2001), "Cyclic Tests of Steel Shear Walls", *Report Number UCB/CE-Steel-01/01,* Department of Civil and Env. Engrg., Univ. of California, Berkeley, August.

Caccese, V. and Elgaaly, M., (1993) "Experimental Study of Thin Steel-Plate Shear Walls Under Cyclic Load", *J. of Str. Engrg*., ASCE, 119, n. 2, pp. 573-587.

CSA, (Canadian Standard Association). (1994). *CAN/CSA-S16.1-94, Limit States Design of Steel Structures.* Sixth Edition, Willowdale, Ontario, Canada.

Driver, R.G., Kulak, Elwi, A. E. and G. L., Kennedy, D.J.L., (1998) "Cyclic Tests of Four-Story Steel Plate Shear Wall", *Journal of St. Engrg.,* ASCE Vol. 124, No. 2, Feb., pp. 112-120.

Rezai, M., Ventura, C. E. and Prion, H.G.L. (2000). Numerical investigation of thin unstiffened steel plate shear walls. *Proceedings*, 12th World Conf. on Earthquake Engineering.

Timler, P. A. (1988) "Design Procedures Development, Analytical Verification, and Cost Evaluation of Steel Plate Shear Wall Structures", *Technical Report No. 98-01,* Earthquake Engrg. Research, Facility, Dept. of Civil Engineering, Univ. of British Columbia, Canada.

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.

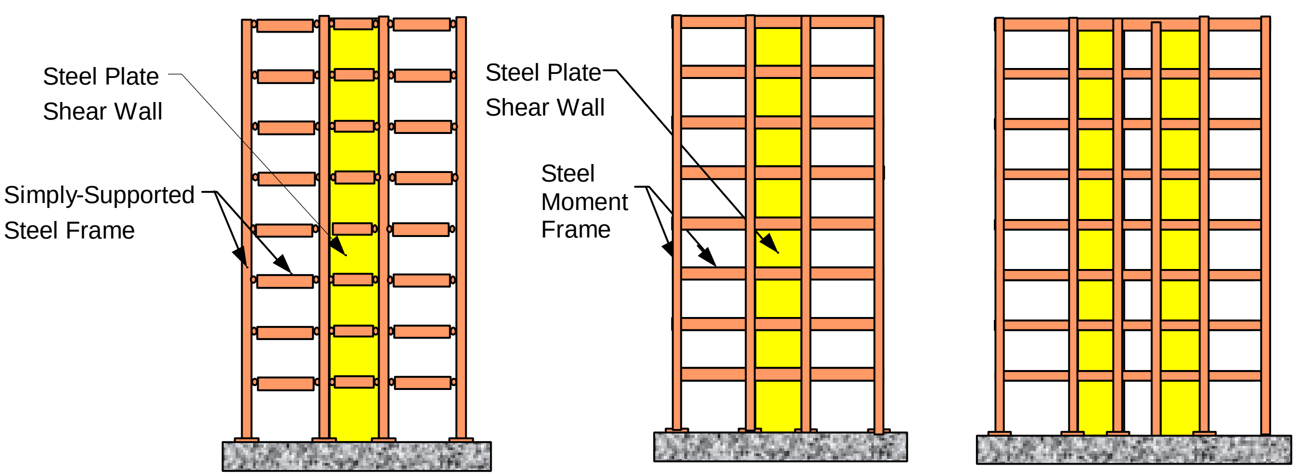
C-16.1. Scope

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

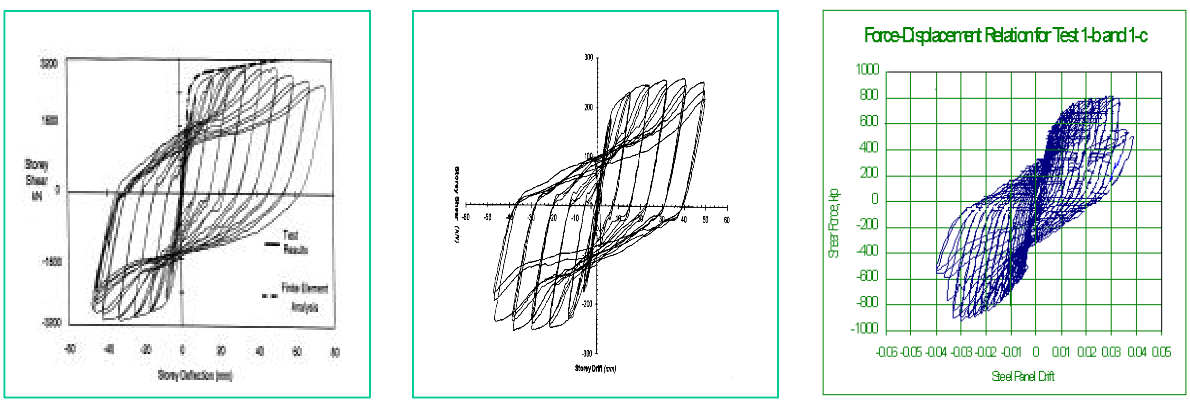
Fig. C-16.1. Typical Steel Shear Wall Systems

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