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Seismic Behavior and Design of Steel Shear Walls

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This report presents information on performance of steel shear walls under seismic effects and their seismic design. Steel shear walls discussed in this report are used to provide lateral strength and stiffness to steel building structures. They have also been used efficiently in seismic retrofit of existing steel or reinforced concrete buildings. Since 1970's structural engineers have used the steel shear walls as lateral load resisting system for new high-rises in highly seismic regions such as California and Japan. This report is prepared to provide the state of the art of the seismic behavior as well as seismic design of steel shear walls. First, some of the important structures in which steel shear walls have been used, are introduced. Then, a summary of the behavior of steel shear walls under cyclic load, in the laboratory as well as during past earthquakes, is presented. Later in the report, current code provisions relevant to steel shear walls are presented and new R-Factor and other design parameters are proposed for steel shear walls. The report also includes a chapter on seismic design of steel shear walls. The design procedures if applied can result in more ductile, economical and better seismic performance. Finally, a number of economical and efficient steel shear wall systems and their details are suggested.

The first edition of this report, which was printed on paper, was released in February 2001. The differences between this electronic edition for the web and the first printed edition are: (a) the results of an additional test of steel shear walls conducted at UC-Berkeley are added to Section 3.5, (b) the equations adapted from the AISC-LRFD Specification are written in the same format as in the just released 1999 AISC-LRFD Specification, (c) any typographical error caught after the first printed version have been corrected in this electronic version and; (d) a page "about the author" has been added to the end of the report.

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This report is dedicated to the memory of the late Professor Emeritus Boris Bresler (1918-2000) of the University of California, Berkeley, who was a leader of structural engineering for more than half a century with many valuable contributions to the field of design of steel structures including authoring a classic textbook on the subject with T.Y. Lin and J. Sclazi.

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The author is a professor of structural engineering, with emphasis on steel structures, at the University of California at Berkeley. He is also a member of the Structural Steel Educational Council (SSEC), Research Council on Structural Connections (RCSC), Earthquake Engineering Research Institute (EERI), American Society of Civil Engineers (ASCE), Structural Stability Research Council (SSRC), Structural Engineers Association of Northern California (SEAONC) and the Council on Tall Buildings and Urban Habitat (CTBUH).

The opinions expressed in this report are solely those of the author and do not necessarily reflect the views of the University of California, Berkeley, the Structural Steel Educational Council or other agencies and individuals whose names appear in this report.

SEISMIC BEHAVIOR AND DESIGN OF STEEL SHEAR WALLS

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Notations and Glossary

A. Notation

In preparing the following notations, whenever possible, the definitions are taken from various references as indicated inside the parentheses whenever applicable.

A Area of cross section.
A_b Area of one bolt.
A_e Effective net area.
A_g Gross area.
A_i The floor area in square feet of the diaphragm level immediately above story.
A_n Net area.
A_{np} Net area of plate.
A_w Shear area of wall=dt_w .
a Height of story in tension field action equations (AISC, 1999).
b Width of unstiffened element.
b_f Width of flange.
C_d Deflection amplification factor .
C_{pr} A factor to account for peak connection strength(FEMA, 2000).
C_s Seismic coefficient given by IBC-2000.
C_v Ratio of plate critical stress in shear buckling to shear yield stress(AISC, 1999).
D The effect of dead load(IBC-2000).
D Diameter of hole.
d Overall width of wall.
d_w Width of shear wall, back-to-back distance of outside flanges of columns.
E Modulus of elasticity.
E The combined effect of horizontal and vertical earthquake-induced forces (IBC-2000).
E_m The maximum seismic load effect (IBC-2000).
F_y Specified minimum yield stress of the type of steel to be used, ksi. As used in the LRFD Specification, "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have yield point). (AISC, 1997) .
F_{ye} Expected yield Strength of steel to be used,(AISC, 1997).
F_{yw} Specified yield strength of steel shear wall.
F_u Specified minimum tensile strength,(AISC, 1997) .
h Clear width of shear wall from column flange to column flange.
h_c Height of column or panel in stiffened shear wall.
I_E The occupancy importance factor given by IBC-2000.
K A factor in earlier seismic codes representing ductility etc.

K Effective length factor in buckling of a column.
k_v Plate buckling coefficient (AISC, 1999).
Q_E The effect of horizontal seismic forces (IBC-2000).
R Response modification factor.
R_{CAN} R-factor in Canadian Codes.
R_d Coefficient representing global ductility (SEAOC, 1999).
R_o Coefficient representing system over-strength (SEAOC, 1999).
R_n Nominal strength. (AISC, 1997).
R_u Required strength. (AISC, 1997).
R_{US} R-factor in US Codes.
R_y Ratio of the Expected Yield Strength F_{ye} to the minimum specified yield strength F_y. (AISC, 1998) .
r Radius of gyration in buckling.
r_{max} Maximum values of r_{max_i} .
r_{max_i} The ratio of the design story shear resisted by the most heavily loaded single element in the story to the total story shear, for a given direction of loading. For shear walls see Section 1617.2.2 of IBC-2000.
S_1 The maximum considered earthquake spectral response acceleration at 1-second period (IBC-2000).
S_{DS} The design spectral response acceleration at short periods (IBC-2000).
T The fundamental period.
t Thickness of element.
t_f Thickness of flange.
t_w Thickness of shear wall or web.
V Shear force, also base shear.
V_n Nominal shear strength of a member or a plate.
V_{ne} Expected shear capacity of a member or a plate.
V_u Required shear strength on a member or a plate.
V_y Shear yield capacity.
W Weight of structure, IBC-2000.
Δ Design story drift. (AISC, 1997).
Δ_u Ultimate displacement.
Δ_y Yield displacement.
α Angle of struts replacing a shear wall.
ϕ Resistance factor.
ϕ_v Resistance factor in shear=0.90. (AISC, 1997) .
ϕ_c Resistance factor for compression=0.85, (AISC, 1997).
λ_c Slenderness parameter for a column, $= (KL/pr)\sqrt{F_y/E}$.
λ_e Limiting slenderness parameter for non-compact shear walls, $= 3.53\sqrt{k_v E / F_{yw}}$.
λ_p Limiting slenderness parameter for a compact element. (AISC, 1997).
λ_r Limiting slenderness parameter for a non-compact element. (AISC, 1997).
λ_{tf} Limit of h/t_w for slender shear walls.
μ Ductility equal to ultimate displacement / yield displacement.

ρ Reliability factor based on system redundancy (IBC-2000).
ρ_i Reliability factor for a given story (IBC-2000).
σ Normal stress.
σ_{cr} Critical value of normal stress in plate buckling (SSRC, 1998).
τ Shear stress.
τ_{cr} Critical shear stress in plate buckling.
Ω_o System over-strength factor.

B. Glossary

In preparing the following glossary, whenever possible, the definitions are taken with permission of the AISC, from the Seismic Provisions for Structural Steel Buildings (AISC, 1998).

Shear Wall. A vertical plates system with boundary columns and horizontal beams at floor levels that resists lateral forces on the structural system.

Connection. A combination of joints used to transmit forces between two or more members. Connections are categorized by the type and amount of force transferred (moment, shear, end reaction).

Design Story Drift. The amplified story drift determined as specified in the Applicable Building Code.

Design Strength. Resistance (force, moment, stress, as appropriate) provided by element or connection; the product of the nominal strength and the resistance factor.

Dual System. A Dual System is a structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment resisting frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear and concrete or steel shear walls or steel braced frames (EBF, SCBF or OCBF); and, (3) each system designed to resist the total lateral load in proportion to its relative rigidity.

Expected Yield Strength. The Expected Yield Strength of steel in structural members is related to the Specified Yield Strength by the multiplier R_y.

Nominal strength. The capacity of a building or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects, and differences between laboratory and field conditions.

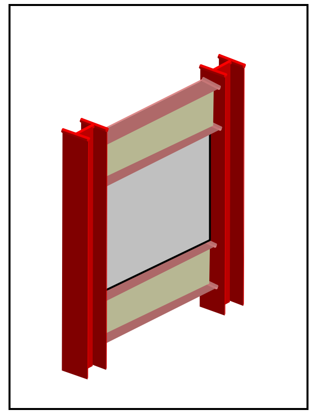
Required Strength. The load effect (force, moment, stress, or as appropriate) acting on a member or connection that is determined by structural analysis from the factored loads using the most appropriate critical load combinations, or as specified in these Provisions.

Slip-critical Joint. A bolted joint in which slip resistance on the faying surface(s) of the connection is required.

Static Yield Strength. The strength of a structural member or connection that is determined on the basis of testing that is conducted under slow monotonic loading until failure.

Structural System. An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

1. INTRODUCTION



The main function of steel plate shear wall is to resist horizontal story shear and overturning moment due to lateral loads. In general, steel plate shear wall system consists of a steel plate wall, two boundary columns and horizontal floor beams. Together, the steel plate wall and two boundary columns act as a vertical plate girder as shown in Figure 1.1. The columns act as flanges of the vertical plate girder and the steel plate wall acts as its web. The horizontal floor beams act, more-or-less, as transverse stiffeners in a plate girder.

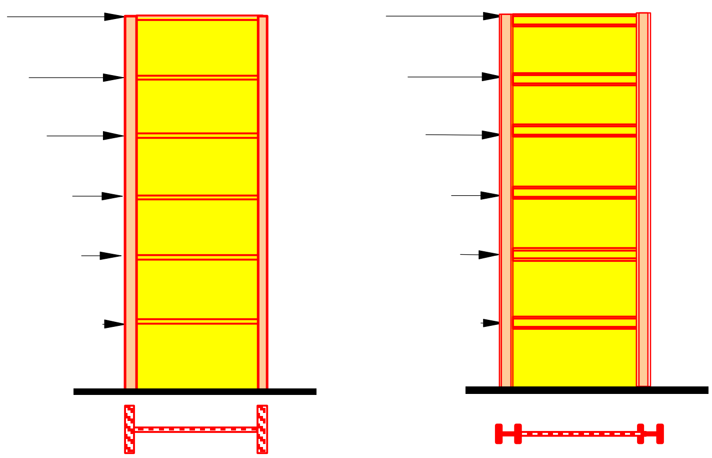


Figure 1.1. A typical plate girder and a steel shear wall

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. Two of these structures have been subjected to relatively large magnitude earthquakes in recent years and have performed very well with minor or no structural damage. In addition to observing behavior of steel plate shear walls during actual earthquakes, their behavior under cyclic loading has been studied by a number of researchers in laboratories in the U.S., Canada and Japan. Although the technology of design and construction of steel plate shear walls have progressed enough, with the exception of Canadian Code (CCBFC, 1995), there is very limited seismic code provisions on steel plate shear walls.

The main objectives of this Steel TIPS report are:

1. To provide information on a number of modern structures where steel shear walls have been used successfully.
2. To summarize the available information on seismic behavior of steel plate shear walls during major earthquakes.
3. To summarize the published results of laboratory tests and other studies done on steel shear walls.
4. To discuss code provisions relevant to seismic design of steel shear walls. Since currently such design code provisions are very limited, a number of design provisions will be proposed that can be used in seismic design of steel shear walls and perhaps after review, modification and refinements by code-writing bodies, and consensus of the profession, some of these provisions can be incorporated into seismic design codes.
5. To present seismic design procedures for steel shear walls.
6. To provide some suggested detailing for steel shear walls that can result in ductile and efficient lateral load resisting systems that are economical and easy to design and construct.

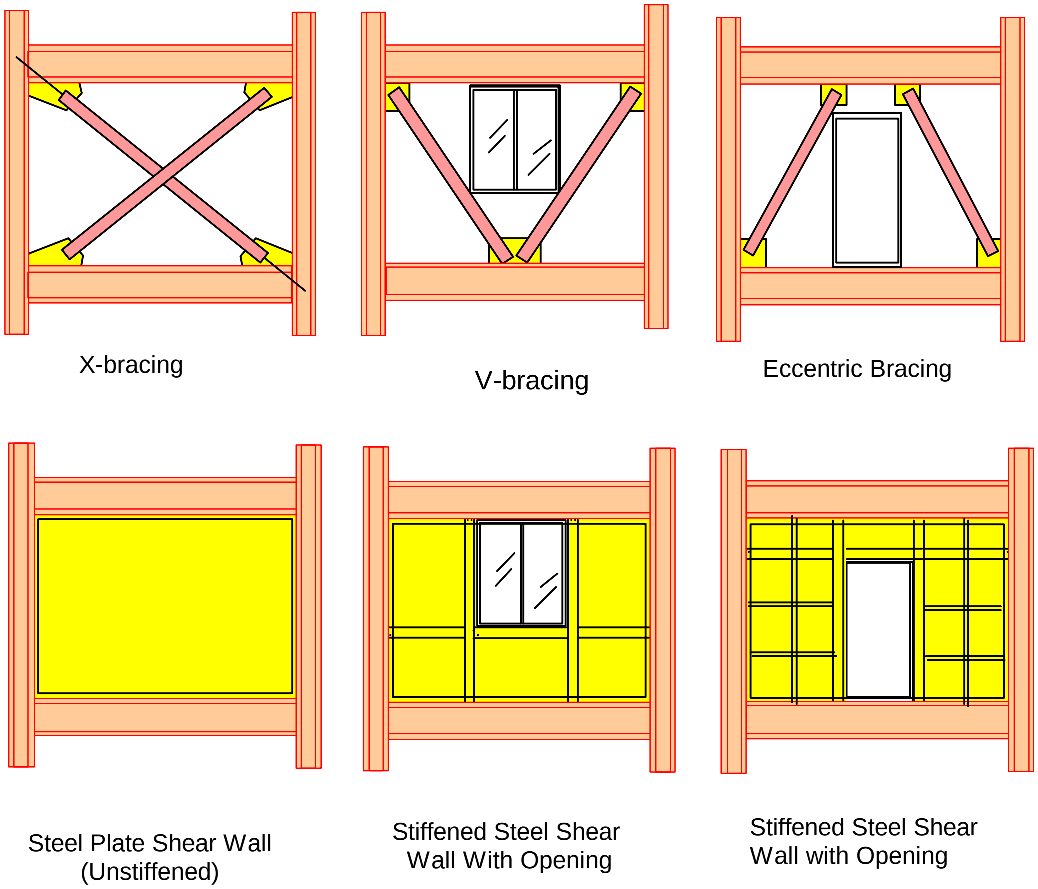


Figure 1.2. Three braced bays (top row) and same bays with steel shear walls

Additional considerations in using steel plate shear walls are:

1. In early applications of steel plate shear walls in the United States, the walls had vertical and horizontal stiffeners. In Japan, almost all of their steel plate shear walls are stiffened. Welding such stiffeners to the wall results in increasing the shear yield strength of the wall. However, in today's steel fabrication shops, welding stiffeners to steel wall can be costly as well as time-consuming. In recent years, the research and testing of realistic specimens have indicated that the steel plate alone, without stiffeners, performs in a very ductile, desirable and efficient manner. As a result, in most applications of steel shear walls in recent years in the United States and Canada, un-stiffened steel plates have been used efficiently and economically.
2. Compared to braced bays, where some window or door openings can be accommodated, in the case of steel shear walls such openings can be accommodated by stiffening the wall around the openings as shown in Figure 1.2. Notice that for un-stiffened steel plate shear walls, to maintain continuity of tension field action, the openings can only be around the

mid-height areas of columns and mid-span areas of the beams. Another solution would be to have two separate shear walls connected to each other by coupling beams as shown in Figure 1.3. This system of shear wall combined with coupling beam can be very ductile and can have very desirable performance (Astaneh-Asl and Zhao, 2000) as discussed in Chapter 4.

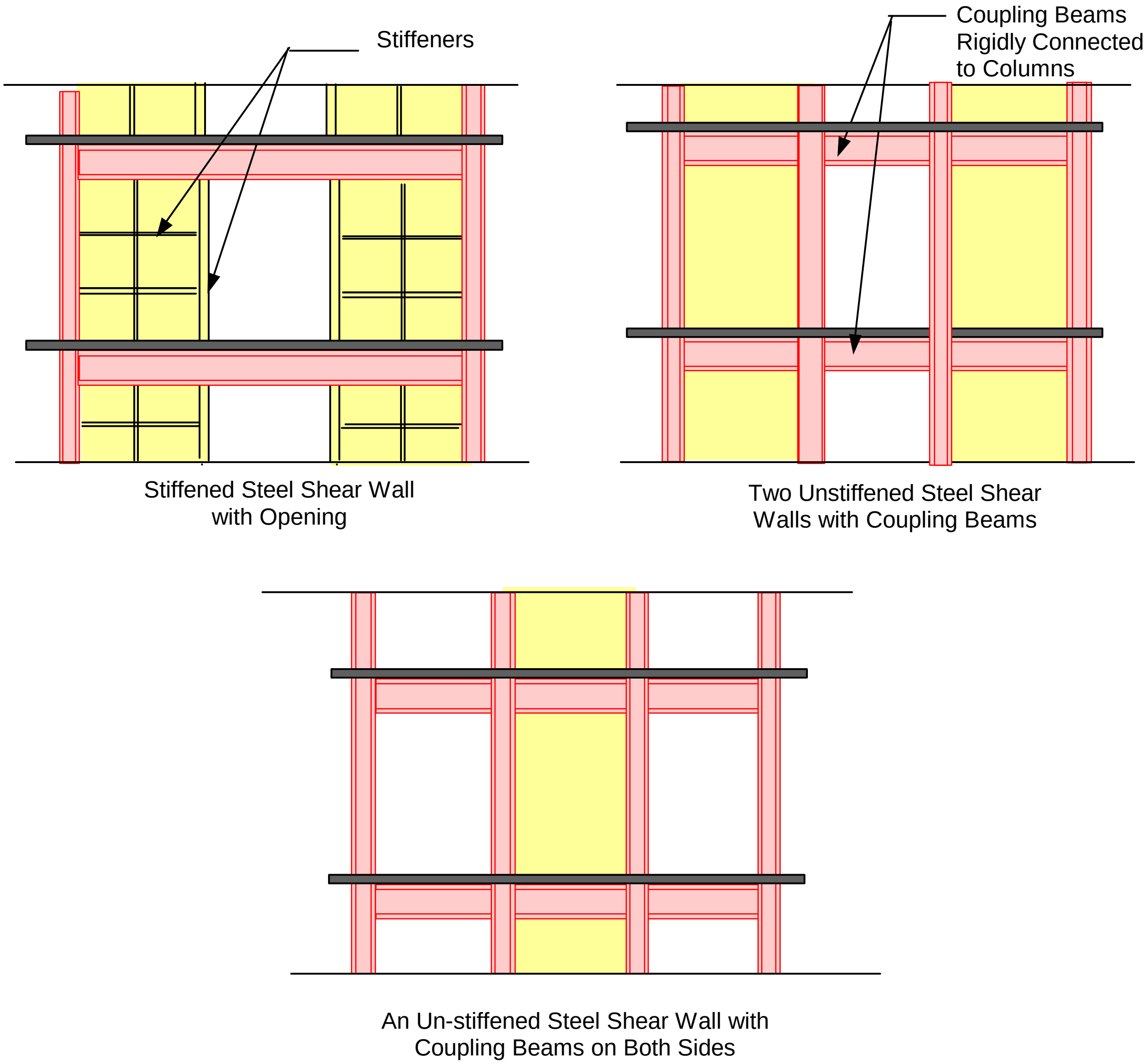


Figure 1.3. Samples of steel shear walls with opening

2. USE OF STEEL SHEAR WALLS AND THEIR SEISMIC BEHAVIOR

Rendering courtesy of
Skilling, Ward, Magnusson, Barkshire

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. The discussion of composite shear walls made of steel plates and concrete cover is left for a future Steel TIPS Issue (Astaneh-Asl, 2001). In the following presentation, if the building with steel shear wall has been subjected to a significant earthquake, its seismic performance is also summarized.

2.1. A 20-story office building in Tokyo, Japan

According to Thorburn et al. (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. Figure 2.1 shows a typical plan. The steel plate wall panels consisted of 9' by 12'-2" steel plates with horizontal and vertical steel channel stiffeners. Figure 2.2 shows the details of steel plate shear wall system. 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.

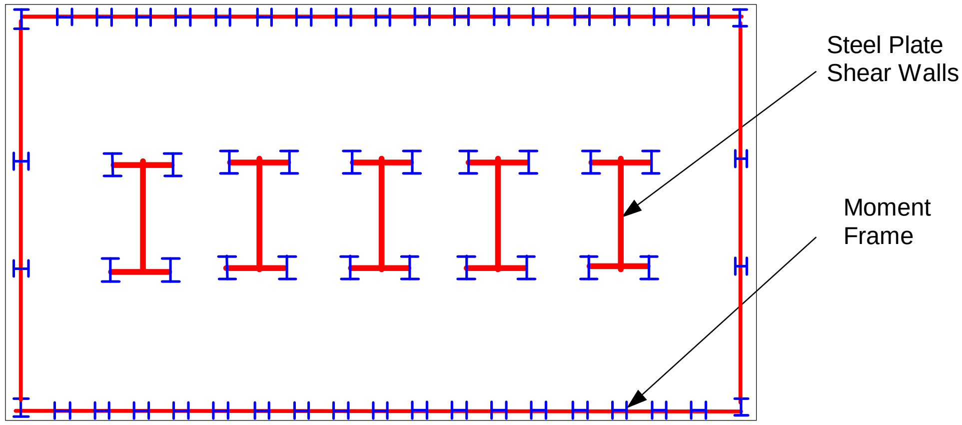


Figure 2.1. Typical floor plan of Nippon Steel Building

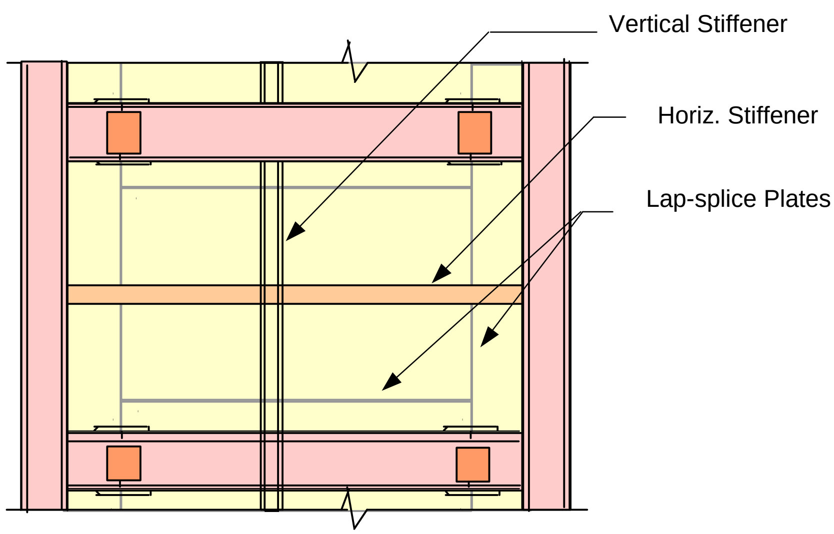


Figure 2.2. Details of steel shear walls used in Nippon Steel Building

2.2. 53-story high-rise in Tokyo

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. Figure 2.3 shows a plan view and elevation of the building. 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.

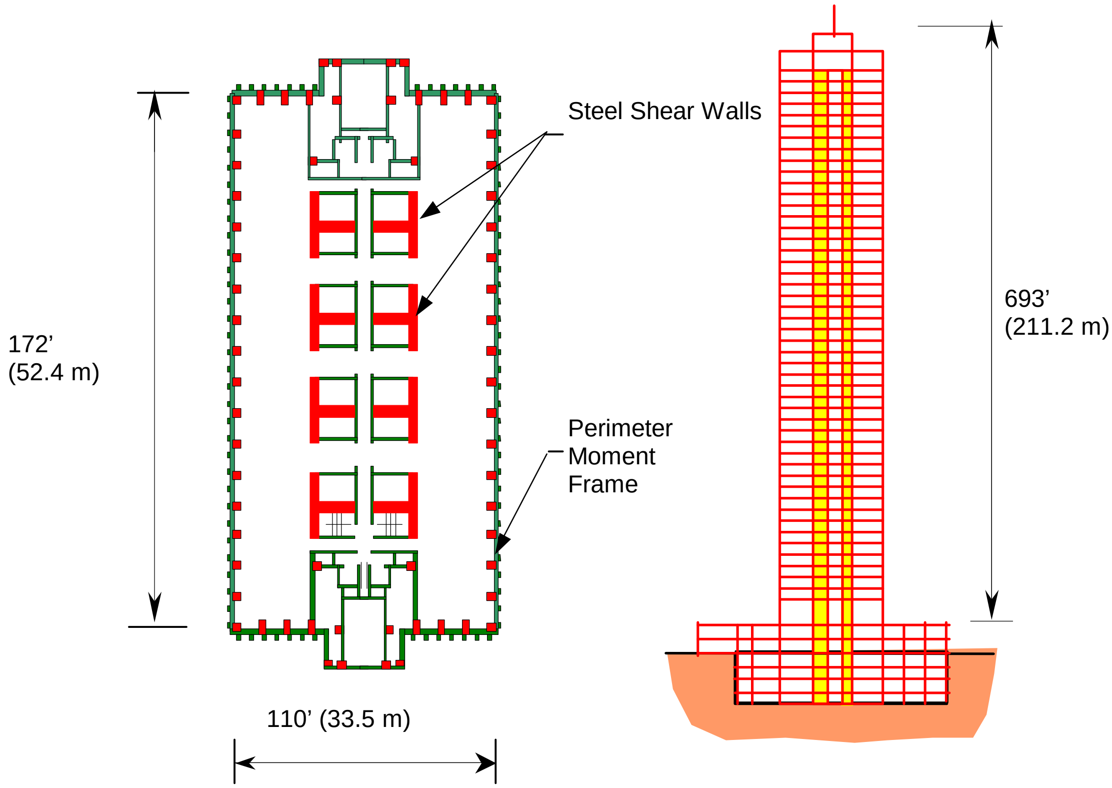


Figure 2.3. Plan, transverse section and a view of the 56-story building in Tokyo

2.3. A 30-story hotel in Dallas, Texas

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. Figure 2.4 shows a view of the building.

Figure 2.4. A View of 30-story building in Dallas

2.4. A 6-story hospital in Los Angeles, California

This structure shown in Figure 2.5 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.

Figure 2.5. A view of Sylmar hospital

In the new Sylmar Hospital, shown in Figure 2.5, 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 2.6. 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 2.6. 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 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.

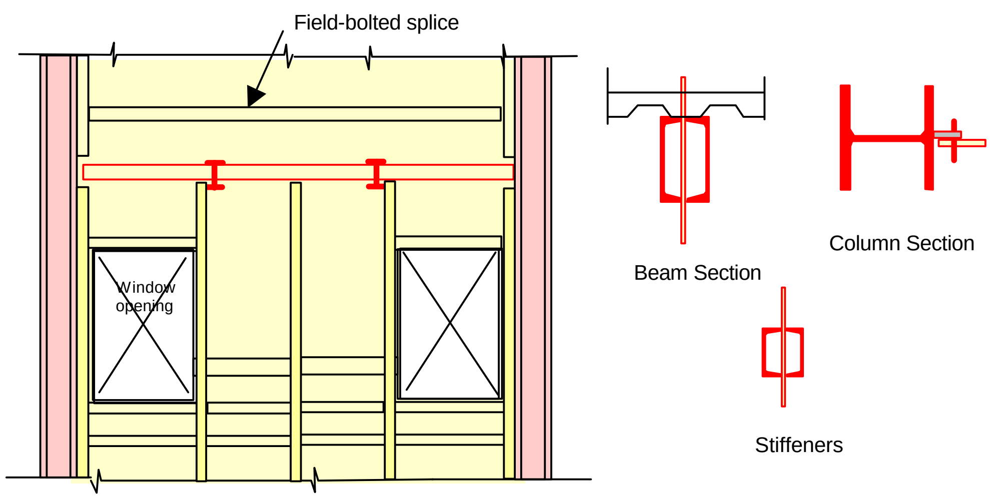


Figure 2.6. A view of stiffened steel plate shear walls of Sylmar hospital

The California Strong Motion Instrumentation Program (CSMIP) has instrumented the Sylmar hospital. Figure 2.7 shows the direction of accelerations measured and Channel Number for each instrument. The 1987 Whittier and the 1994 Northridge earthquakes shook the structure and valuable records on response of the structure were obtained. Figure 2.8 shows data recorded by the CSMIP instruments in this building during the 1994 Northridge earthquake. The accelerations at roof level were more than 2.3g while the ground acceleration were 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).

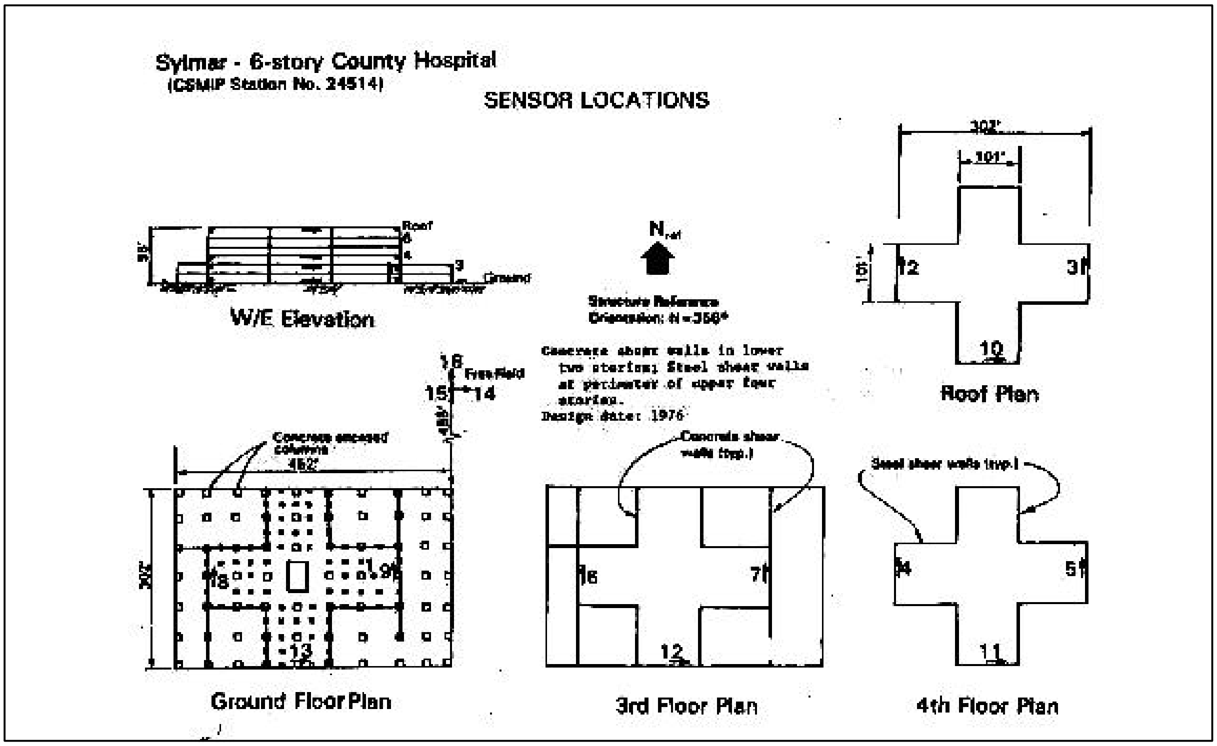


Figure 2.7. The instrumented Sylmar hospital, (CSMIP, 1994).

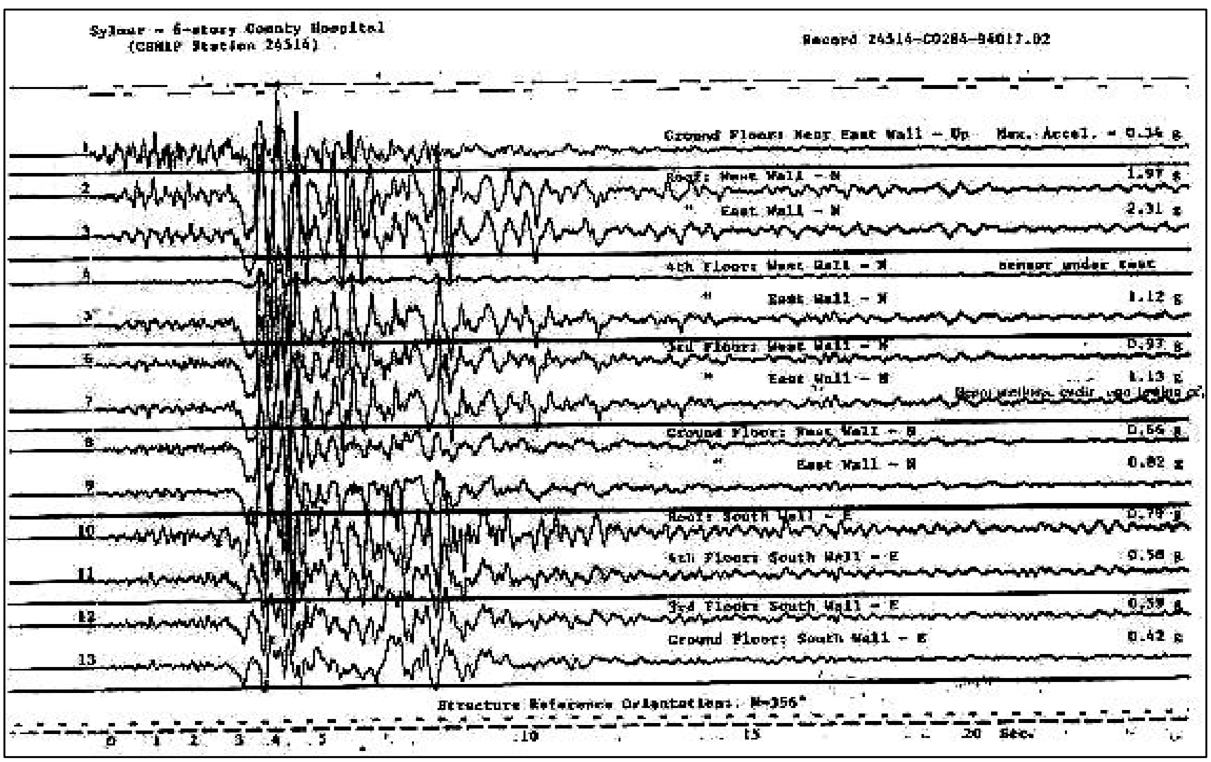


Figure 2.8. Records obtained from instruments in Sylmar hospital, (CSMIP, 1994)

2.5. A 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. 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. Figure 2.9 shows framing plan and typical frames. The author visited this building about two weeks after the 1995 Kobe earthquake and found no visible damage.

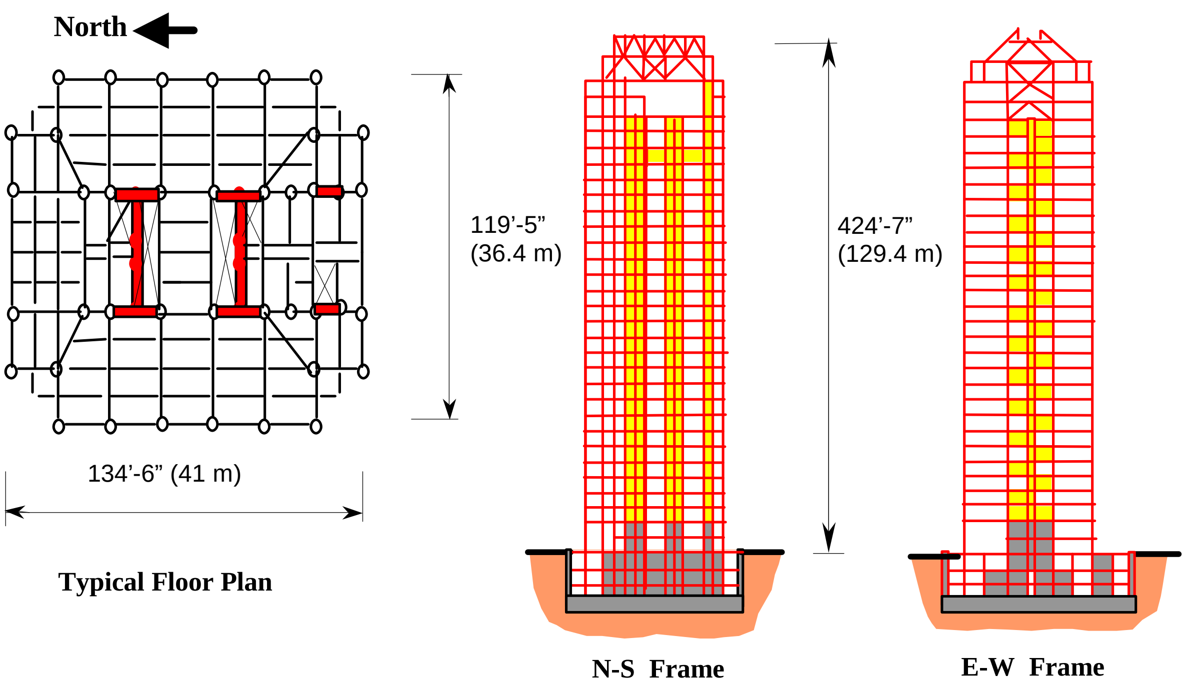
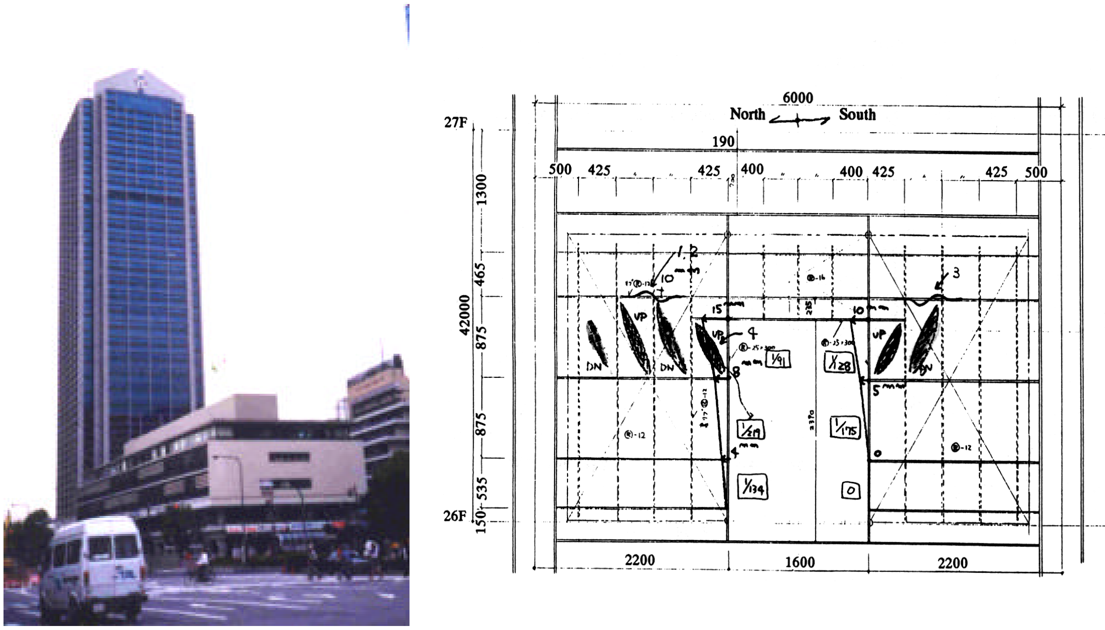


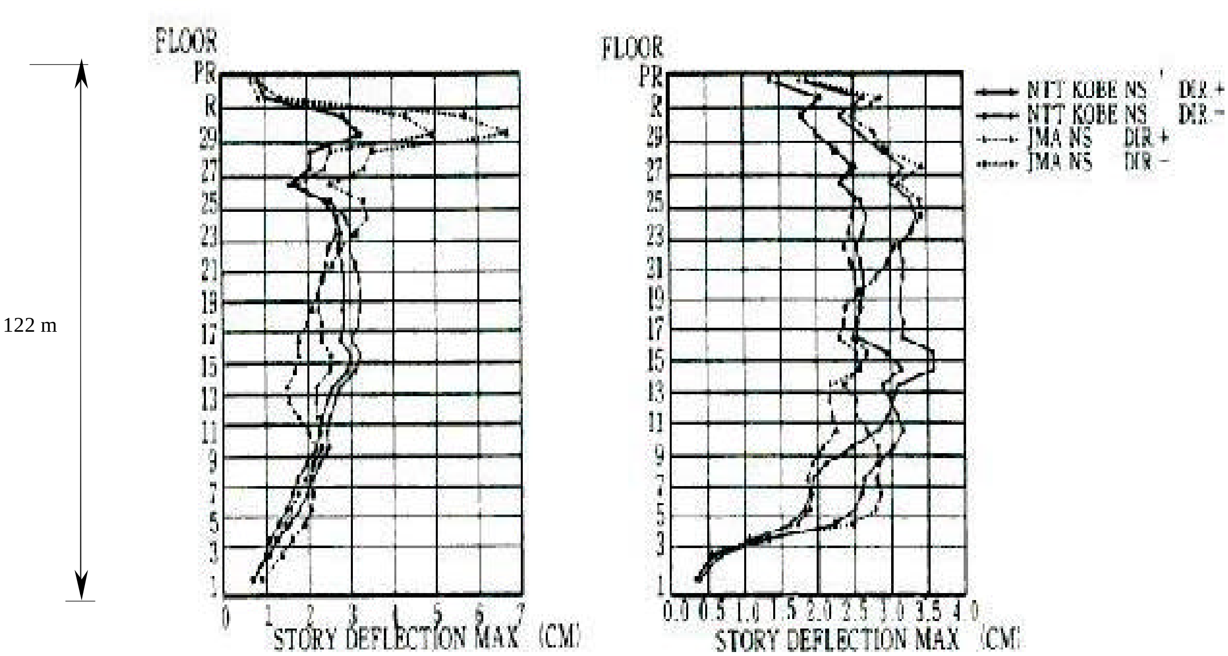
Figure 2.9. Structure and a view of 35-story Kobe building

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 2.10 also shows a sketch of damage to the shear wall at 26th floor. 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. Figure 2.11 shows results of story drifts in NS and EW direction when an inelastic model of the structure was subjected to two Kobe ground motions (AIJ, 1995). The maximum inter-story drift is about 1.7% in 29th floor of the NS frame.



(Photo by M. Kanada, from Kanada and Astaneh-Asl, 1996), (From: Fujitani et al., 1996) and (AIJ, 1995)

Figure 2.10. A view of the 35-story building and damage to 26th floor shear walls



(From: Fujitani et al. , 1996)

Figure 2.11. Results of drift values in NS and EW frames of Kobe high-rise),

2.6. A 52-story residential building in San Francisco, California

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 structure is designed by Skilling, Ward, Magnusson, Barkshire of Seattle and is currently under construction. The building is a residential tower and when completed will have 48 stories above ground and four basement parking levels. A rendering of the building and a typical floor plan are shown in Figure 2.12.

Figure 2.12. A rendering of Century building

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.

2.7. A 22-story office building in Seattle, Washington

A view of this building is shown in Figure 2.13. At this writing, (summer of 2000) the structure is being designed by Skilling Ward Magnusson Barkshire. A typical floor framing consists of typical steel deck/concrete floors supported on wide flange beams and columns.

(Rendering courtesy of Skilling Ward Magnusson Barkshire)

Figure 2.13. A rendering of Seattle building

The lateral load resisting system consists of a core with four large concrete filled tubes on its corners and steel plate shear walls and coupling beams connecting the tubes to each other in one direction and steel braced frame in the other.

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.

2.8. Use of "Low-Yield" steel shear walls in Japan

In recent years, "low-yield point (LYP) " steel plates have been developed in Japan and used successfully as steel plate shear walls. Low yield steel has a yield point of about half of A36 steel with much greater ductility and ultimate elongation of more than twice that of A36 steel. It has been demonstrated in Japan that such steel can be very effectively used as energy dissipating element of the structure. Later in Chapter 3, more information on low-yield steel, its properties and cyclic behavior is provided. In the following the use of low yield steel plates as steel shear walls in Japan is discussed. Figure 2.14 shows a building where the low yield point steel is used in the core elevator / stairwell shaft of the building. The LYP panels are stiffened and have been shop welded and field bolted.

(photos: Nippon Steel, Japan)

Figure 2.14. A view of building with Low Yield Point (LYP) steel plate shear walls and a close-up of the walls

Figure 2.15 shows another example of recent application of low-yield point (LYP) steel plate shear walls in a 31-story building in Japan. According to Yamaguchi et al. (1998), the LYP steel used in this structure had a 2% offset proof stress (yield point) of 11.6 –17.4ksi, a tensile strength of 29-43.5 ksi and percent of elongation at fracture exceeding 50%.

The walls were approximately ¼ inch to one inch thick and about 14'-9" by 9'-10" stiffened by horizontal and vertical stiffeners. The prefabricated LYP wall units were connected to the boundary beams and columns using friction bolts. The walls were designed to remain elastic under wind load but yield under "Level 1" earthquake.

The designers report that as a result of using low yield steel the drift values decreased about 30%. From material given in reference (Yamaguchi et al, 1998) it could be deducted that apparently the reason for arranging the LYP steel shear walls in an alternate pattern is to reduce bending effects. It appears that such an alternate pattern can also prevent gravity load from accumulating in the walls. As a result such walls are primarily subjected to shear and relatively small bending effects. Moment frames carry the overturning moments due to lateral loads.

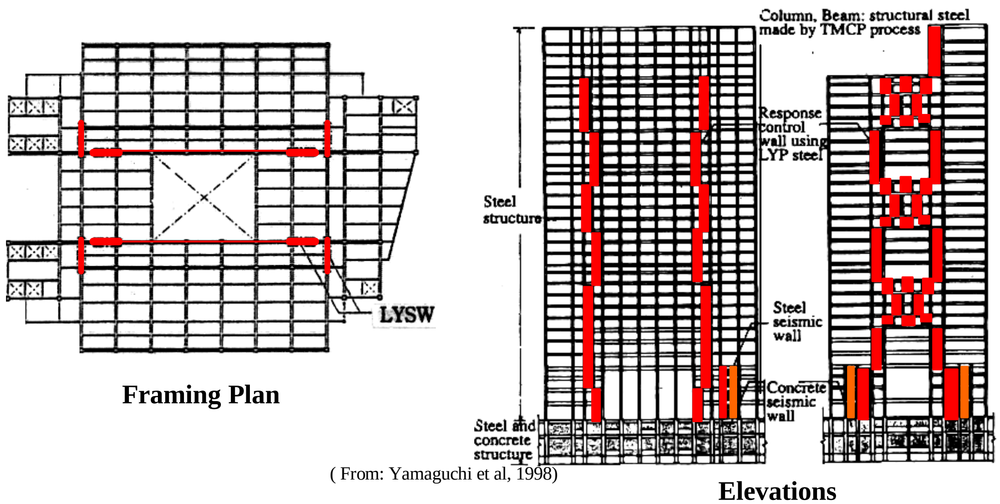


Figure 2.15. Framing plan and elevations of 31-story building in Japan using low-yield steel shear walls

2.9 Use of "Viscous Wall Damper " steel shear walls in Japan

The Oiles Corporation of Japan has developed a system of steel plate shear wall that basically acts as a viscous damper. Figure 2.16 shows a sketch of this system as well as the Media City Shizuoka building, one of the structures where this system is used. Studies by Shimoda et al (1996) shows potential of this system to improve seismic behavior and reduce seismic effects considerably.

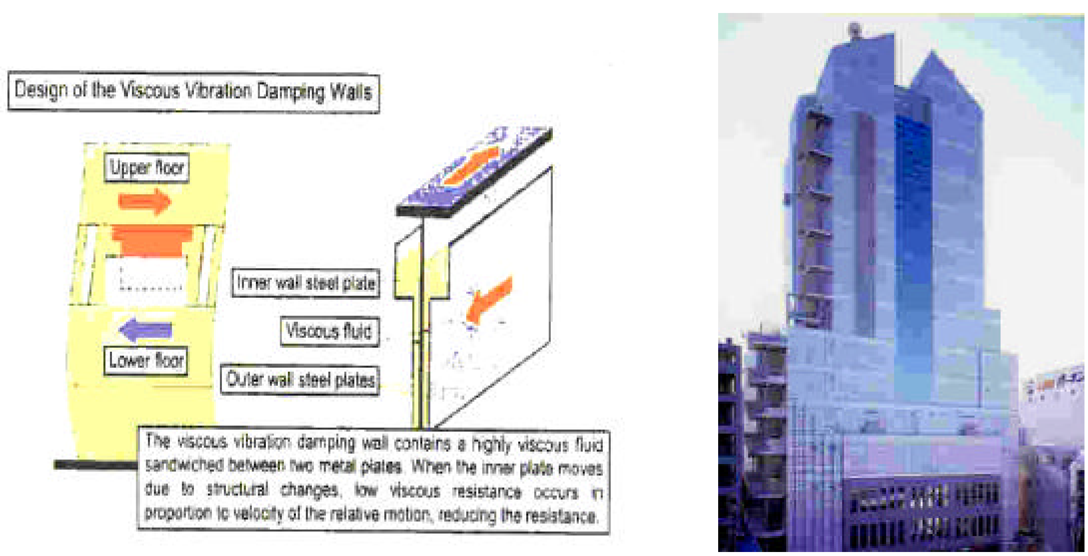
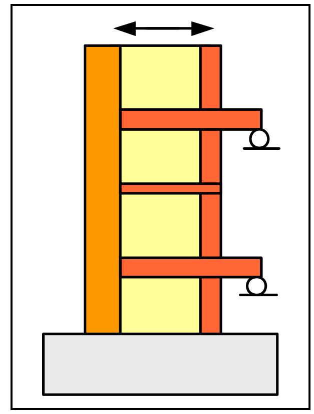


Figure 2.16. The "Oiles Viscous Wall Damper" system and one of the buildings where it was used

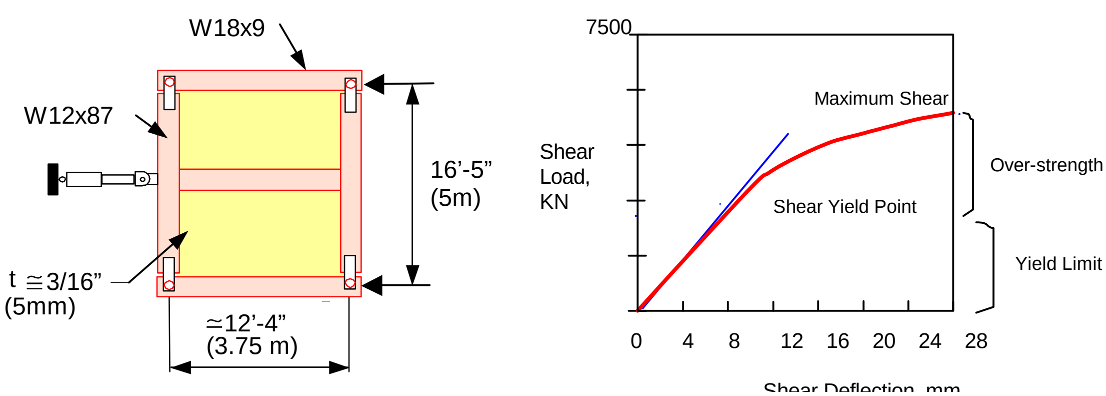
3. LABORATORY TESTS OF STEEL SHEAR WALLS



A number of researchers in United States, Japan, Canada and United Kingdom have studied behavior of steel shear walls. This chapter provides brief summaries of experimental studies and testing of shear walls.

3.1. Tests of Steel Plate Shear Walls in Canada

Researchers at the University of Alberta, (Timler and Kulak, 1987), (Kulak, 1991), and (Driver et al., 1996), have conducted monotonic and cyclic tests of un-stiffened steel plate shear walls. A summary of these tests is given in the following. Figure 3 shows the specimen and the load displacement curve for static monotonic test by Timler and Kulak (1983). The load displacement curve indicates a ductile behavior and significant over-strength. The specimen exhibited a ductility exceeding 4.0. Earlier, Thorburn et. al. (1983) 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.



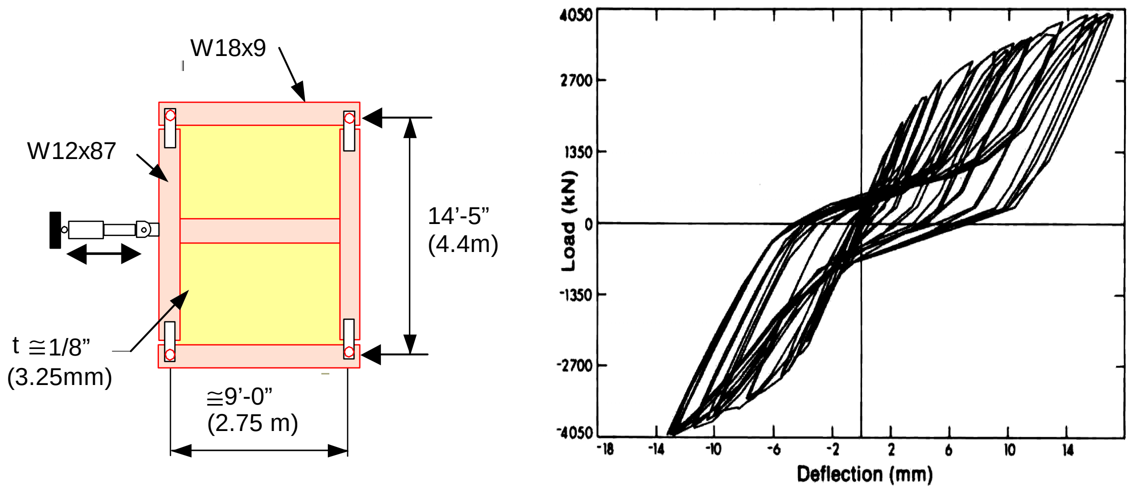
Test Specimen

Load vs. Displacement

(Note: The curve traced from original plot in Timler and Kulak, 1983)

Figure 3.1. The University of Alberta's first test (static load)

Figure 3.2 shows the specimen tested by Timler and Kulak (1983) under cyclic loading. The figure 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.

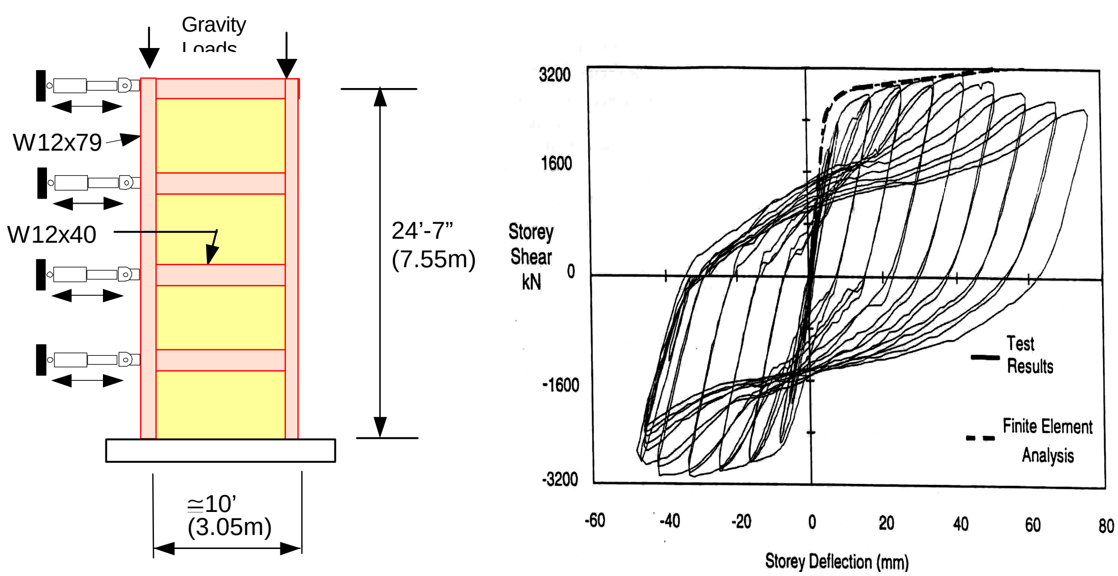


Test Specimen

Load vs. Displacement
(Curves from: Timler and Kulak, 1983)

Figure 3.2. The University of Alberta's second test (cyclic load)

Driver et al (1996 and 1998) have reported the results of cyclic testing of a four-story steel shear wall specimen. The specimen, shown in Figure 3.3, was a ½-scale representation of a dual system with steel plate shear wall welded to a special moment-resisting frame.



Test Specimen

Load vs. Displacement
(Curves From: Driver et al, 1996)

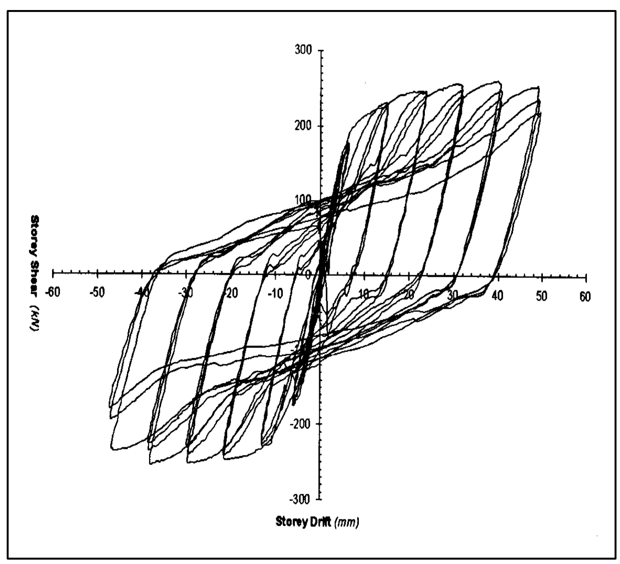
Figure 3.3. University of Alberta test set-up and a sample of hysteresis behavior

Figure 3.3 shows cyclic response of the first floor steel shear wall panel. The failure mode was fracture of left column at the heat-affected zone of weld connecting the column to the base plate. The researchers (Driver et al, 1998) 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 (Lubell, 1997), Rezai, 1999) and (Rezai et al.1988 and 2000). In these studies, cyclic shear loads were applied to two single story specimens. Figure 3.4 shows one of the specimens after the test and hysteresis behavior of the specimen. 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.

(Photo: Curtsey of C. Ventura))

Test Specimen

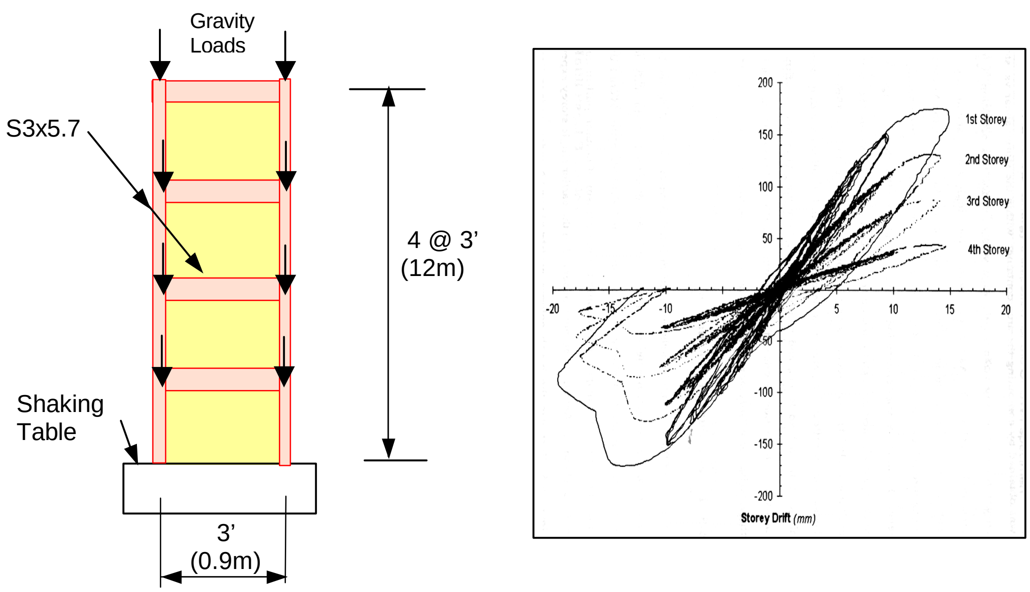


(Curves from: Lubell, 1997)

Load vs. Displacement

Figure 3.4. One of the two Univ. of British Columbia specimens with its hysteresis loops

In the shaking table tests, (Lubell, 1997), a four-story specimen representing 30% scale model of inner core of a residential building was used. Figure 3.5 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. The members were S3x5.7 (Canadian S75x8). The roof level beam was an S8x23 (Canadian S200x34). 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 3.6 shows force-deformation hysteresis loops for the first floor. For full details of the cyclic tests, refer to Lubell (1997) and Rezai (1999).



(Curves from: Lubell , 1997)

Specimen on Shaking Table   Hysteresis Response

Figure 3.5. The specimen tested at the Univ. British Columbia and its response

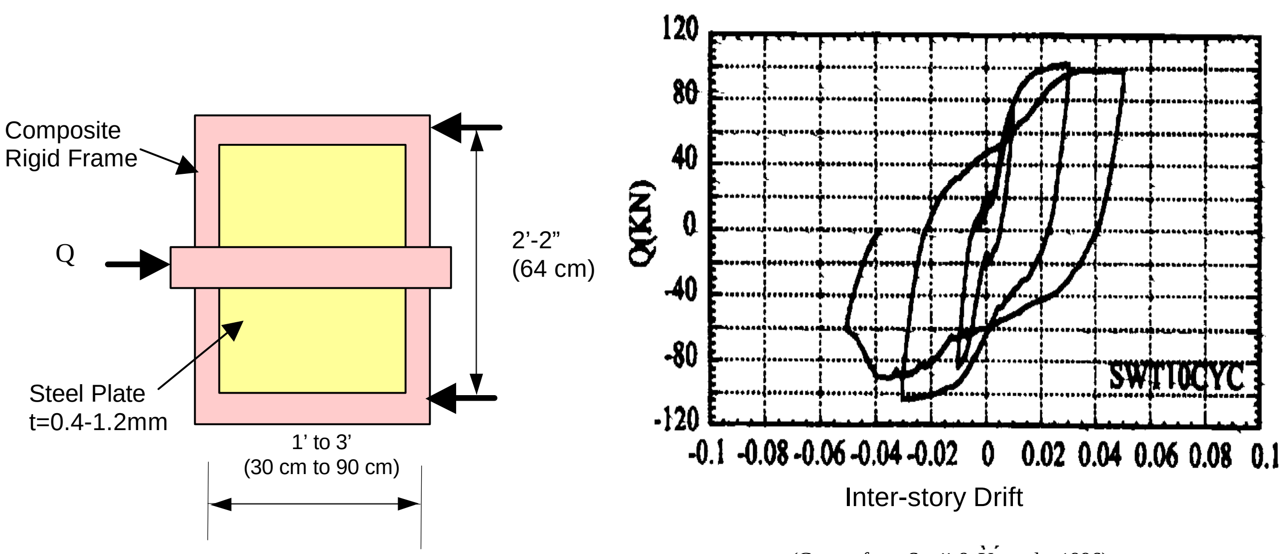
3.2. Tests of Steel Plate Shear Walls in Japan

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 1/4 –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 behavior 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 discussed in Section 2.5 of this report. 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 3.6 shows a typical specimen and hysteresis loops. All specimens showed pinching of hysteresis loops due to buckling of compression field.



(Curves from Sugii & Yamada, 1996)

Specimen

Hysteresis Behavior

Figure 3.6. A typical test specimen and typical hysteresis loops for 1 mm plate

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 behavior of steel shear wall panels made of "low yield" steel. Figure 3.7 shows stress-strain curves for mild steel (similar to A36), high strength steel (Similar to A572, Gr. 50) and the "low yield steel" developed by Nippon Steel in Japan. In general yield point of current low yield steel is about half of A36 steel and its ultimate strain is more than twice as much as that of A36 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 3.7 shows a sample of such curves.

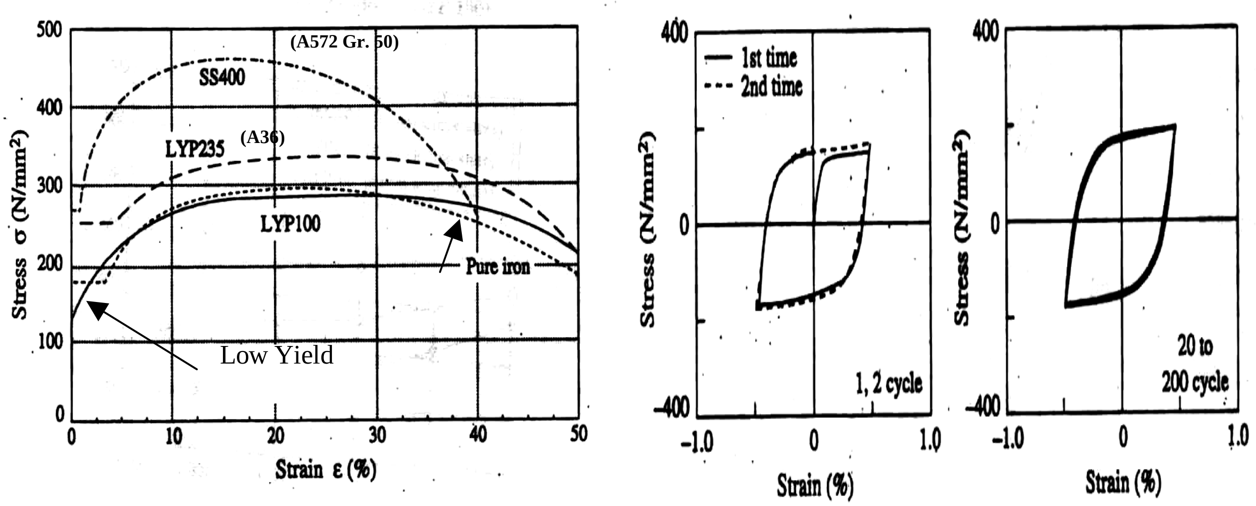
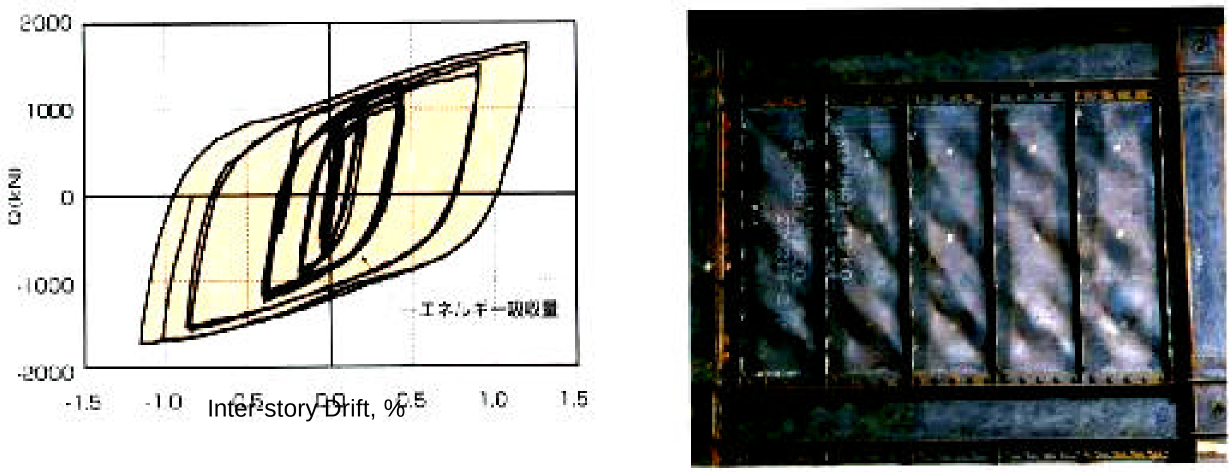


Figure 3.7. Stress-strain curves and hysteresis behavior of low- yield steel
(Nippon Steel, 1998)

Figure 3.8 shows typical hysteresis behavior of specimens tested in this program sponsored by Nippon Steel. 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 3.8 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 (see Section 2.8 of this report).



(Figure from Nippon Steel Publications)

Figure 3.8. Typical specimen, set-up, table of specimens and the hysteresis loops

3.3. Tests of Steel Plate Shear Walls in United Kingdom

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 3.9. The specimens were small-scale with dimensions of steel panels b and d in Figure 3.9 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 in Roberts (1992) shows specimens reaching a ductility of more than seven 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) as shown in Figure 3.10.

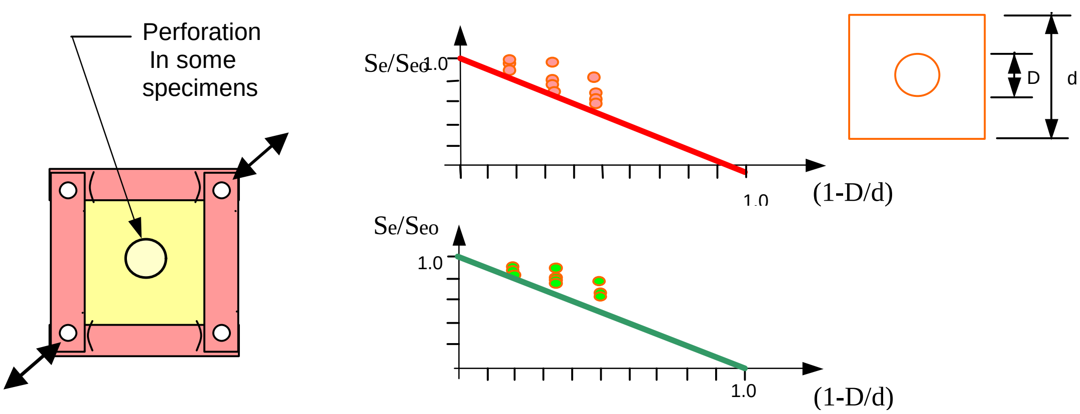
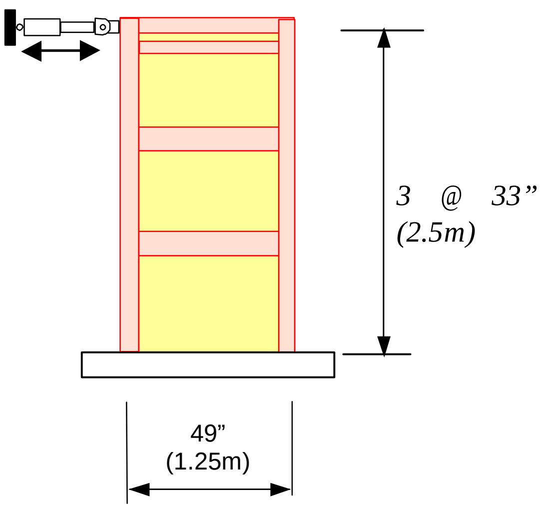


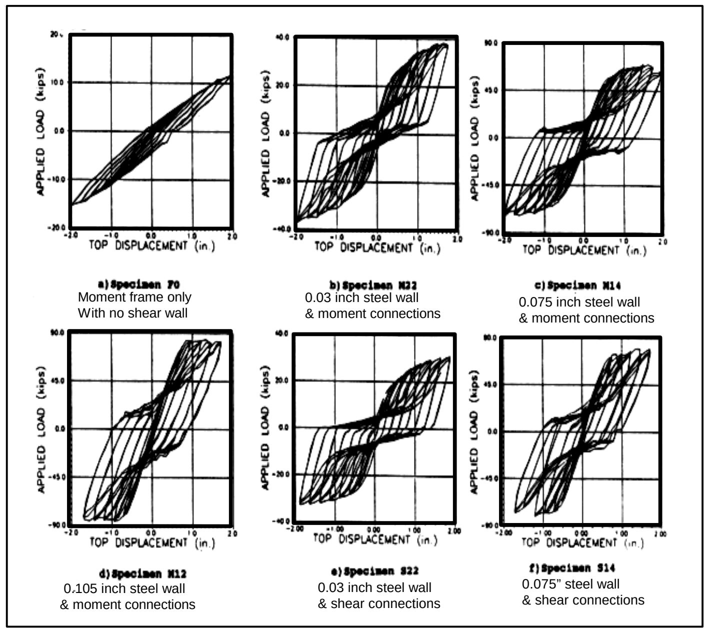
Figure 3.9. Specimens tested in U.K. and the effect of perforation on strength and stiffness of steel plate panels (Roberts, 1992)

3.4. Tests of Steel Plate Shear Walls in the United States

In the United States, Elgaaly and his research associates, (Caccese et al, 1993), (Elgaaly and Caccese, 1993), 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. Figure 3.10 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.



Test Specimen



Load vs. Displacement

(Curves from Elgaaly and Cassese, 1993).

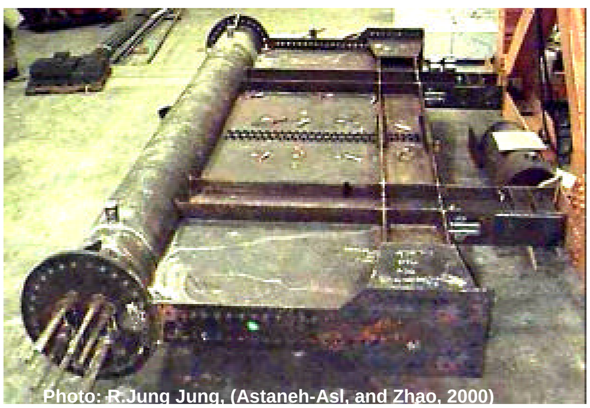
Figure 3.10. Test set-up and hysteresis behavior of specimens

Based on the behavior of these six specimens, Elgaali and Caccese (1993) 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 presented in Chapter 5.

3.5. Current 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 TIPS report (Astaneh-Asl, 2001). In the following, the discussion is limited to the steel plate shear wall tests at UC-Berkeley (Astaneh-Asl and Zhao, 2000).

The test specimens for steel plate shear wall tests were two 1/2-scale, 3-story steel shear wall frame assembly shown in Figure 3.11.



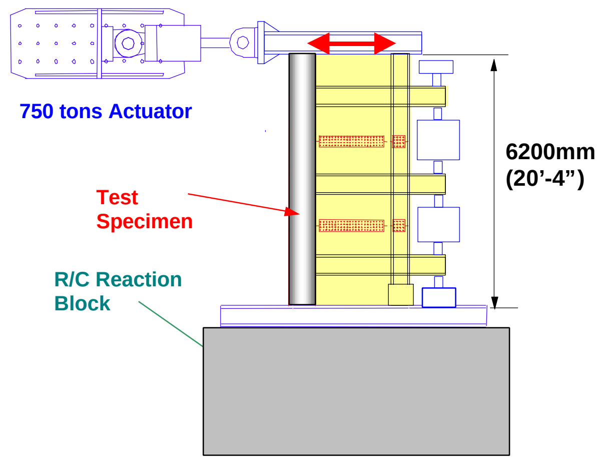
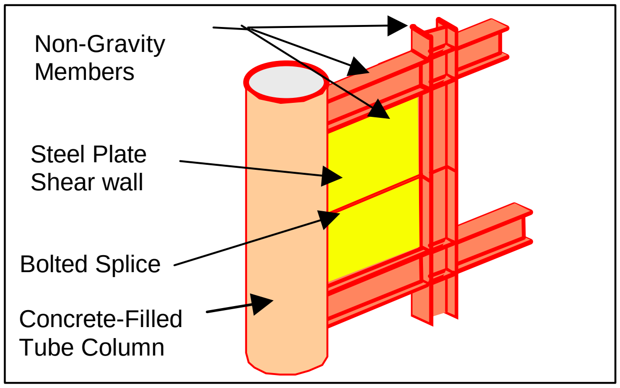


Figure 3.11. Typical specimen and test set-up
(Astaneh-Asl and Zhao, 2000).

At this writing (July 2001) two specimens have been tested. Specimens One and Two The first specimen, shown in Figure 3.12, had height-to-width ratio of about 1.5 while the height-to-width ratio of the wall in Specimen Two was 1.0. The specimens were half-scale realistic representatives of the steel shear wall-moment frame (dual) system used in high-rise structures. Figure 3.13 shows this steel shear wall system. A number of structures with this type of steel shear wall have been designed by Skilling Ward Magnusson Barkshire and constructed on the West Coast of the United States. The most important building with this type of wall is a 51-story high –rise currently under construction in San Francisco (see Section 2.7 of this report). In the following, a brief summary of behavior of the first specimen is provided. Figure 3.13 shows basic details of the system being tested and studied by Astaneh-Asl and Zhao (2000).

Specimen One behaved in a very ductile and desirable manner. Up to inter-story drifts of about 0.6%, the specimen was 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, the WF column developed local buckling. The specimen 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. Figure 3.13 shows the specimen at the end of the test (Astaneh-Asl and Zhao, 2000).



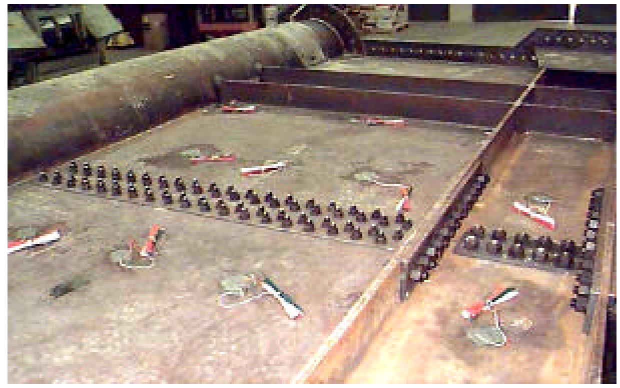


Figure 3.12. Components of the tested system and bolted splice

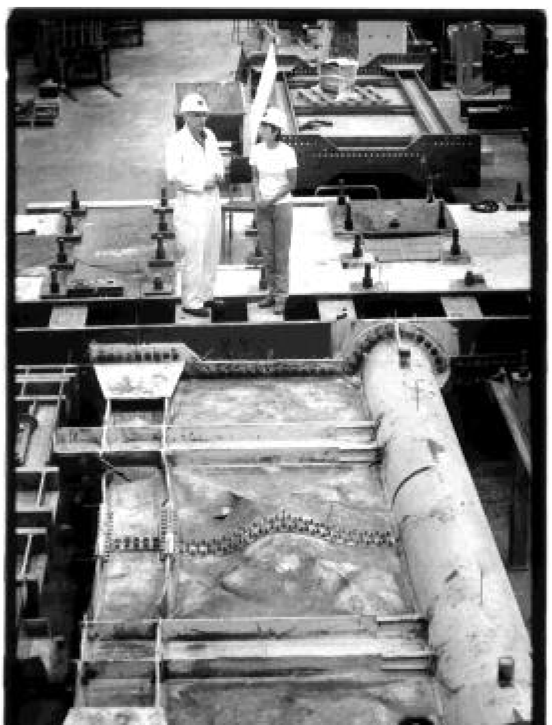
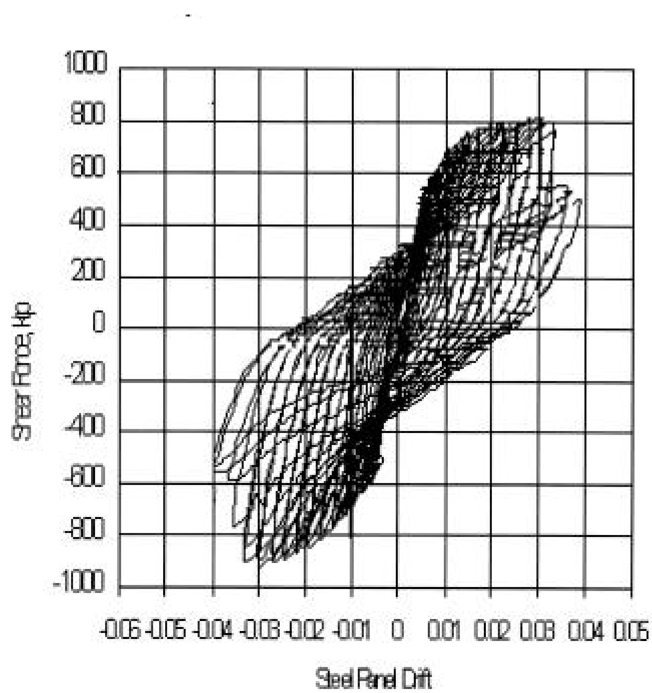


Photo by Bruce Cook. Loaned from Office of Public Affairs, College of Engineering, University of California, Berkeley



(From: Astaneh-Asl and Zhao, 2000)

Figure 3.13. Specimen UCB-1 at the end of the test
(Astaneh-Asl and Zhao, 2000)

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 clearly visible on the steel plate shear walls as shown in Figure 3.14. The specimen could tolerate 29 cycles, out of which 15 cycles were inelastic. The specimen reached an inter-story drift of more than 2.2% and maximum shear force of 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 75 % of the maximum shear force reached in previous cycles (1,225 kips). Since capacity had dropped below 80% of maximum strength, the specimen was considered failed and the testing stopped. Figure 3.15 shows the hysteresis loops for the walls in first floor and second floor of this specimen. More information on these tests can be found in Astaneh-Asl and Zhao (2001).

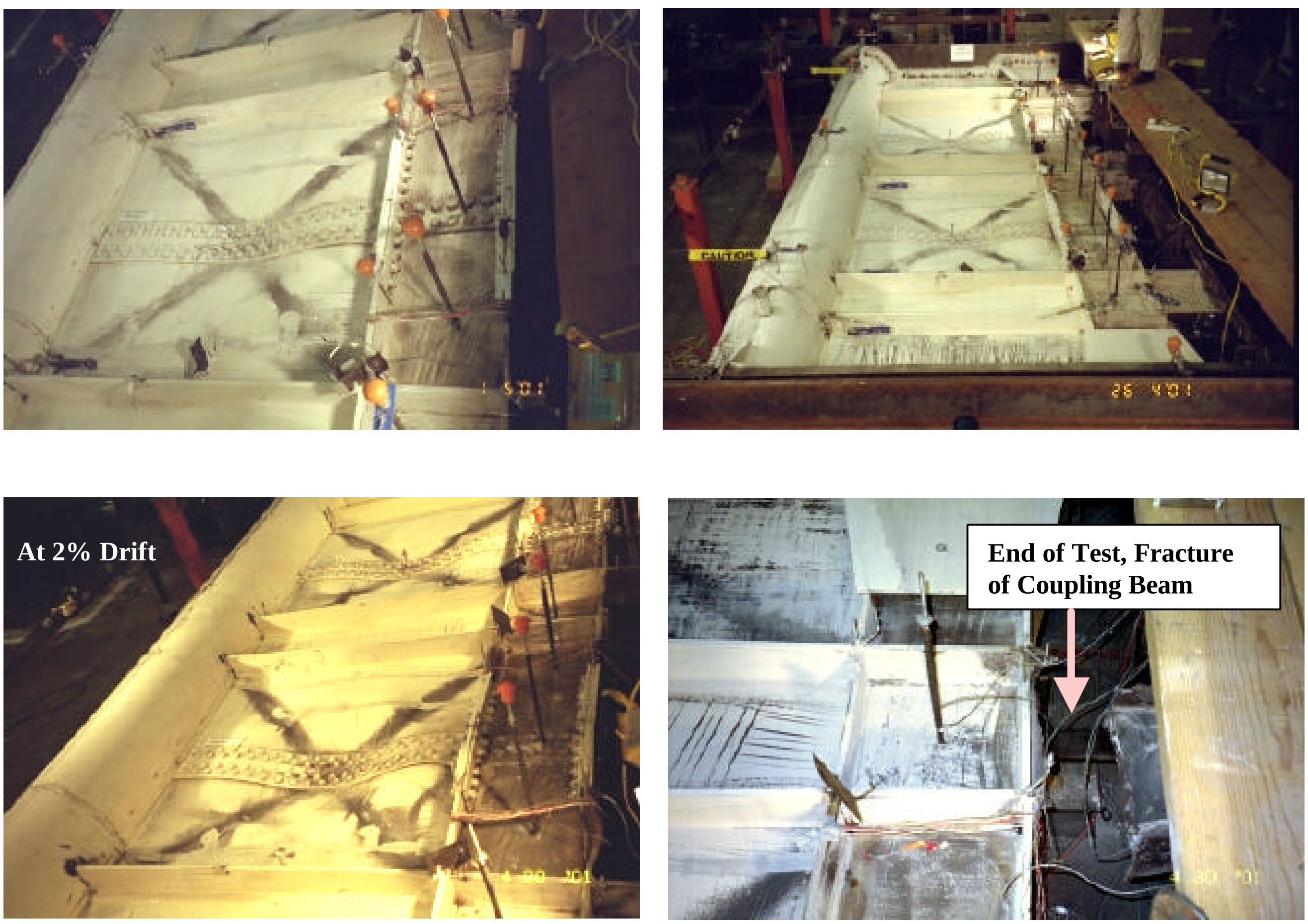
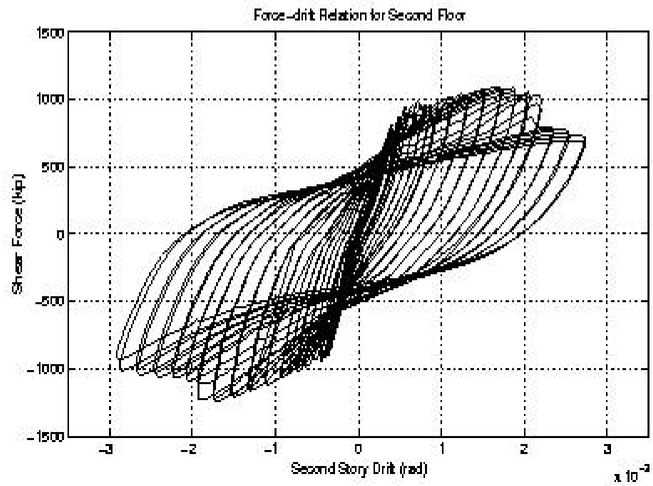


Figure 3.14. Views of Specimen Two during the test and at the end of the test (Astaneh-Asl and Zhao , 2001)



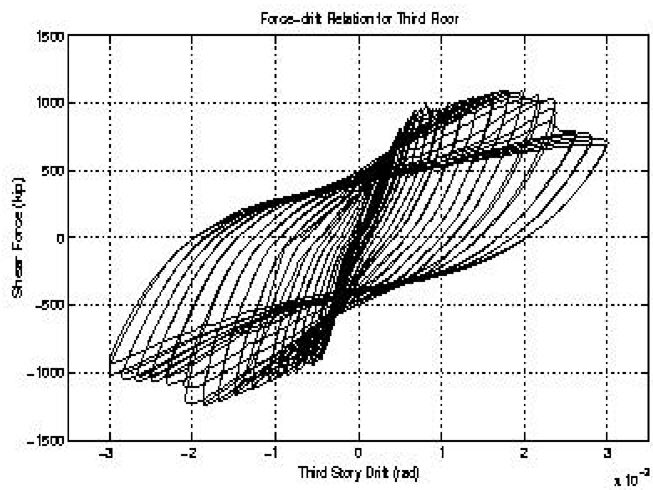
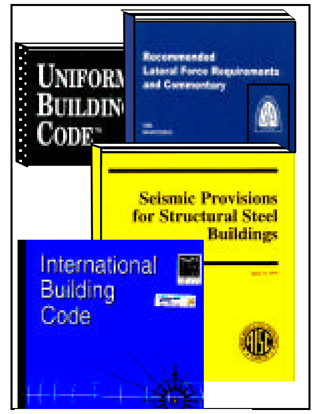


Figure 3.15. Shear force-drift Hysteresis loops for two floors of Specimen Two
(Astaneh-Asl and Zhao, 2000)

4. CODE PROVISIONS



Currently, there is considerable information in the literature and in the US codes that can be used for a rational seismic design of steel shear walls. However, the current seismic codes in the United States do not provide specific values for a number of seismic design parameters for steel shear walls. In particular, US codes do not have specific values for seismic design parameters such as response modification factor R and system over-strength factor Ω_o for steel plate shear wall systems. Also, provisions regarding detailing of steel shear walls are almost non-existent in US codes. Following sections provide a summary of current seismic design provisions in the US codes relevant to steel plate shear walls. In addition, whenever US codes do not provide a key parameter or an important provision, the author has proposed a conservative value of the parameter.

This chapter discusses code provisions primarily from UBC-97 (ICBO, 1997), IBC-2000 (ICC, 2000), SEAOC Blue Book (SEAOC, 1999) and AISC Seismic Provisions (AISC, 1997). The reader is assumed to be familiar with at least one of the UBC-97, SEAOC-99 or IBC-2000 codes and the AISC-97 Seismic Provisions. The code provisions quoted here are for discussion only. In actual seismic design, the users should refer to the actual code document. The author would like to caution the user that the design recommendations proposed in this chapter are for information only. Anyone using such information takes full responsibility for their use.

4.1. Code Provisions Relevant to Seismic Design of Steel Shear Walls

Currently in highly seismic areas of the US the structural engineers frequently use Uniform Building Code (ICBO, 1997). A few months ago, the first edition of the International Building Code, IBC-2000 was released (ICC-2000). Since IBC-2000 is more refined and updated compared to UBC-97, it was felt that as more and more jurisdictions adopt the IBC-2000, it is hoped that it would replace UBC-97 in the coming years. Therefore, in the following sections, the code provisions of the IBC-2000, relevant to steel plate shear walls, are discussed. Whenever appropriate, information and provisions from other seismic design codes such as the National Building Code of Canada (CCBFC, 1995), Recommended Lateral Force Requirement and Commentary (SEAOC-99) and the AISC-Seismic Provisions (AISC, 1997) are also discussed.

4.2. Establishing Earthquake Loads for Steel Plate Shear Wall Systems Using the US Codes

The UBC-97, SEAOC-99 and IBC-2000 have seismic load effects E and E_m that involve information related to the structural system. E and E_m are used in IBC-2000 (as well as in other US codes) in load combinations that are specific to seismic design. Values of E and E_m are given as follows. These are Equations 16-28, 29 and 30 of the IBC-2000.

$$E = \rho Q_E \pm 0.2 S_{DS} D \tag{4.1}$$

$$E_m = \Omega_o Q_E \pm 0.2 S_{DS} D \tag{4.2}$$

In the above equations, negative sign should be used for the second term whenever the gravity and seismic effects counteract. For definition of terms in all equations in this report, see Notations on Page iv. All terms in the above equations, with the exception of ρ, Q_E and $\Omega_?$, are independent of the structural system used. Therefore, for steel plate shear wall system, the parameters that are independent of the system should be established the same way as for any other structural system such as braced frame or moment frame systems. The system-dependent parameters and their values for steel plate shear walls are discussed in the following.

4.2.a. Value of *r* ***for steel plate shear walls:*** The parameter ρ, is a reliability factor based on the system redundancy and is given in IBC-2000 (as well as in UBC-97) as:

$$\rho_i = 2 - \frac{20}{r_{\max_i}\sqrt{A_i}} \tag{4.3}$$

Where, $r_{\max_i}$ and A_i for steel shear walls are defined by IBC-2000 (and UBC-97). For definition of these and other terms see Notations in Page iv of this report.

4.2.b. Value of Q_E (and R-factor) for steel plate shear walls: The term Q_E , represents the effects of horizontal seismic forces. In establishing Q_E , if “Equivalent Lateral Force” procedure of the code is used, first base shear V has to be established. Most seismic design codes have a procedure to establish V. The IBC-2000 provides the following equation for V:

$$V = C_s W \tag{4.4}$$

and;

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} \tag{4.5}$$

For definition of terms in all equations in this report, see Notations on Page iv. All terms in the above equations, with the exception of R, the response modification factor, are independent of the seismic-force-resisting system. Design codes in general provide values of R for most common structural systems. The IBC-2000 (as well as UBC-97 and SEAOC-99) provides values of R for more than 70 different seismic-force-resisting systems. However, currently US codes do not give any value of R for steel shear walls.

Currently. the National Building Code of Canada (CCBFC, 1995), in its “non-mandatory” Appendix M, has specific provisions on “Design Requirements” for unstiffened steel plate shear walls including R factors. The Canadian code discusses only unstiffened relatively thin steel plate shear walls welded along their boundaries to beams and columns. The Canadian code (CCBFC, 1995) provides a value of R equal to 4.0 (in Canadian notation) for steel plate shear walls within a special moment frame. When Canadian R of 4.0 is converted to equivalent value with US code terminology it becomes approximately 8.0.

The R factors in the codes have evolved over the years from earlier parameter, K. In early days of seismic design, structural engineers understood correctly that during a major or even a moderate earthquake, many elements of a structure could yield and deform and dissipate energy of earthquake. This very important yet relatively complex phenomenon results in reduction of seismic forces in a structure compared to the case of the structure remaining fully elastic. Since, even today, most structures are analyzed using elastic analysis methods, such methods result in seismic forces much greater than actual forces that will be developed in the structure. To estimate the actual seismic forces, in early days of modern seismic design, the elastic forces were multiplied by a K factor which was generally a number less than 1.0. The K factors were established for most common systems intuitively and consensually by the structural and earthquake engineering community. Later, K factor was replaced with R_w (for working stress level design) and then with R factor for factored load design.

Although no specific research has been done on identifying all parameters affecting R, by studying the data from performance of structures during earthquakes, laboratory test results and analytical studies, it appears that R factor depends on primarily ductility, over-strength, period of vibration and redundancy in the system (ATC, 1995). In addition other parameters affecting R appear to be vulnerability of gravity load carrying system in case of excessive inelasticity, possibility of progressive collapse in case of local failure, positive or negative contributions of non-structural elements, fracture behavior of the material of the structure, characteristics of the ground motion, properties of the supporting soil, dynamic interaction of the ground, interaction of the foundation system and the structure as well as damping in the system.

Currently, most values of R, given in the codes still have their roots in the K factors of early days of seismic design. Other R factors added in recent years, for example R factors for composite systems, were established by code committees using engineering judgment, intuition and the R factor already established for other structural systems that would have similar seismic behavior to the new system. This may not seem very "scientific" but in the opinion of this author, since it brings in the collective knowledge, experience and wisdom of the structural engineering community in designing structures and observing their performance during earthquakes and in laboratories, the approach is rational. It appears that in design it may be better to have a less accurate method based on reliable and extensive past experience than a very precise method but using limited data from a few tests or worse from pure analysis. Considering many complex parameters affecting R, which are not well understood and established in a reliable manner yet, a purely mathematical approach to establishing R at this time may not be the most reliable approach.

In recent years, several attempts have been made to develop a rational basis for establishing R factors instead of relying fully on intuition and engineering judgment. The reader is referred to Applied Technology Council publication ATC-19, (ATC, 1995) and SEAOC Blue Book (SEAOC, 1999) for more information on these methods and how one can establish R factors for new systems such as steel plate shear walls not included in the current codes.

The Seismology Committee of SEAOC has broken R factor into two separate parts (SEAOC, 1999):

$$R = R_o \, Rd \qquad (4.6)$$

Where, Ro represents the over-strength portion of R and R_d represents the reduction in seismic force due to inelastic actions in the structure. One can establish values of Ro and R_d by considering structural system characteristics listed by SEOAC (1999) as:

1. *Observed and/or predicted system performance under strong ground motion.*
2. *Level of inelastic deformation capability.*
3. *Vulnerability of vertical load carrying system.*
4. *Degree of redundancy in lateral force carrying system.*
5. *Multiplicity of lines on resistance, such as back-up frames*

(Excerpt from SEOAC Blue Book, 1999, Ref. (SEAOC, 1999)

The ability of steel plate shear walls to meet the above-listed characteristics is briefly reviewed below.

1. Observed and/or predicted system performance under strong motion:

Chapter 2 of this report provided a summary of the available information on performance of steel plate shear wall buildings during past earthquakes. Although only two buildings with steel shear walls were severely shaken by the past earthquakes, these buildings have performed extremely well. The damage to both buildings was very minor or no damage to the structure and no damage to gravity load carrying systems.

The laboratory cyclic and shaking table tests, as summarized in Chapter 3, have indicated very ductile and desirable behavior for the steel plate shear wall systems. The cyclic tests were primarily conducted by Takanashi, Elgaali, Kulak, Ventura, Astaneh-Asl and their research associates. The specimens in these tests ranged from one to four-story and were scaled models. The largest specimens so far have been three-story, ½-scale (Astaneh-Asl and Zhao, 2000). The observed behavior and data from these tests, briefly summarized in Chapter 3, clearly indicate that in all cases the steel shear wall systems demonstrated two main characteristics of a "very desirable" system. The two characteristics were: (a) the systems had sufficient *stiffness* and *strength* to resist low and moderate earthquakes as well as wind loads while remaining almost elastic and; (b) the systems had high degree of redundancy, were very ductile, capable of dissipating significant amount of energy while undergoing very large number of cyclic inelastic load reversals beyond what might be expected during a major seismic event.

2. Level of inelastic deformation capability:

The performance of steel plate shear walls during actual earthquakes, discussed in Chapter 2, and the cyclic and shaking table tests of steel shear walls, summarized in Chapter 3, have indicated that these systems can tolerate relatively large number of inelastic cycles. The ductility of the tested systems expressed in terms of $\mu = \Delta_u / \Delta_y$, was relatively high and in the order of 5 to 8. The test results indicate that from ductility point of view the steel plate shear walls posses' larger ductility than all other structural systems that are currently listed in seismic design codes and have R-factors assigned to them.

3. Vulnerability of vertical load carrying system:

Data on the actual behavior of steel shear walls during earthquakes and their behavior when subjected to cyclic loading in laboratories, summarized briefly in Chapters 2 and 3, indicate that in these systems the inelasticity is primarily in the non-gravity carrying element of the system, which is the steel wall itself. Such yield behavior is very desirable since the gravity load carrying systems remaining undamaged, can carry gravity load during and after the seismic event and prevent collapse.

In structures where the boundary columns of the shear wall system are carrying gravity load, failure of columns during an earthquake can result in unacceptable vulnerability of the gravity load carrying system. This is a problem for almost all shear wall systems, steel and concrete, where boundary columns have to resist the axial forces due to seismic overturning moments in addition to their gravity load. To prevent failure of gravity load carrying boundary columns, a "hierarchical" seismic design procedure should be followed. In the hierarchical design procedure the order of failure modes are such that the failure modes that can harm gravity load carrying elements do not occur until more desirable failure modes such as yielding of tension field areas in the wall have occurred. The yielding of non-gravity carrying elements can be designed to act as a fuse to limit the amount of force that can be transmitted to more critical gravity load carrying elements such as boundary columns. In Chapter 5, a " hierarchical" seismic design procedure for steel plate shear walls is developed and suggested by the author. If the proposed procedure is used it is expected to result in inelasticity mainly in the form of yielding of steel plate, taking place in non-gravity carrying elements which act as plastic fuses to protect gravity load carrying elements such as columns from buckling or tension yielding.

Another approach to ensure that gravity load carrying elements such as boundary columns are not vulnerable is to design boundary columns of the shear wall very strong with almost no possibility of inelasticity or avoid gravity load from being transferred to boundary columns altogether. The SWMB has successfully implemented the concept in developing their steel plate shear wall system. A. Astaneh-Asl and Zhao (2000) recently tested the system and found it to be quite ductile. Chapter 4 of this report has a summary of behavior of this system.

4. Degree of redundancy in lateral force carrying system:

Degree of redundancy of a structure can be considered at two levels: (a) at the global structural level by studying how many lateral force resisting systems there are in the structure and how well they are distributed in the plan and over the height of the structure and (b) at the local level of lateral load resisting system itself by studying how much redundancy and force distribution capability the lateral force resisting system has within a given story and over the height. The issue of global redundancy currently is handled by codes through introduction of ρ factor and is not incorporated into R factors directly. The ρ and code provisions to establish it for a steel plate shear wall system was discussed earlier in this chapter in Section 4.2.a.

The issue of redundancy within the lateral force resisting system is very important and influences the R factors. Tests of one to four-story steel plate shear wall systems, summarized in Chapter 3, have indicated that steel shear walls have very high degree of redundancy by the fact that plates are highly indeterminate systems. When diagonal areas of a steel plate shear wall yields, during subsequent cycles the yielded area steel resists the force but the extra force is shed to neighboring areas of the plate and the yielding continues to spread to more areas of the plate. During the tests conducted by Astaneh-Asl and Zhao (2000) at very late cycles of behavior and after a few inches of the steel plate on the diagonal tension axis had fractured, the wall was still carrying the applied load. This was due to the fact that the tensile diagonal force was resisted by the areas of the plate off the diagonal axis that still were elastic or yielded but not fractured.

5. Multiplicity of lines on resistance, such as back-up frames

When steel plate shear wall is not part of a "dual" system, the structural engineer generally models the frames to have pin connections. In reality, the frames have shear connections and studies of shear connections subjected to seismic effects (Liu and Astaneh-Asl, 2000) clearly have indicated that these connections act more or less as semi-rigid and posses considerable moment capacity in the order of 20%-70% of plastic moment capacity of connected beams. Although in current practice structural engineers do not include such moment capacities in design, in reality the capacities exist and contribute to lateral load-resisting system. In case of steel plate shear walls that are part of a frame with simple connections, the simple frames acting as semi-rigid frames provide a good back up system for the steel shear walls. These back-up systems are not considered in design but are there as additional, albeit with relatively less capacity, lateral force resisting systems.

In the aftermath of the 1994 Northridge earthquake, A. Astaneh-Asl (1995) studied the damaged steel moment frames and concluded for the first time, and to the disbelief of a few structural engineers that even after all welded moment connections of the study structures were cracked, still the structures were not in danger of immediate collapse due to aftershocks or other earthquakes. Other researchers later substantiated these findings as well. The main reason for the stability of damaged moment frames was attributed by Astaneh-Asl et al. (1998) to contribution of shear connections acting as semi-rigid connections with the floor slab and producing semi-rigid frames made of these connections, floor system and gravity columns outside the moment frames. Such a system acts as a very good back–up system for steel plate shear walls as they do for moment frame systems.

For the case of dual shear wall systems, steel shear walls are combined with moment frames to resist lateral forces. Obviously, due to high elastic stiffness of the steel shear walls; during the elastic range of behavior and even when wall has some limited yielding, the wall will resist the bulk of the lateral force. As yielding spreads more and more into the off diagonal areas of the wall and the stiffness of the steel wall decreases sufficiently, the stiffness of the moment frame becomes comparable and steel wall and moment frame share the lateral force in a more balanced manner. This phenomenon has been clearly observed and recorded during the recent steel shear wall tests by Astaneh-Asl and Zhao (2000).

It is interesting to note that if the steel plate shear wall is placed inside the moment frame (instead of being parallel to it and on a different frame line), the presence of the wall plates at the corners of the moment frame create gusset plate like brackets that can help reduce the rotation demand on the connections of the moment frame. Also, when shear wall is placed inside the moment frame, the sharing of lateral force between the wall and moment frame is direct. In this case, there is no need for stiff and strong floor diaphragm for transfer of load between the shear wall and the moment frame. Because of these two advantages, in the opinion of the author, dual systems where steel plate shear walls are placed within the moment frame are preferred and expected to exhibit good behavior.

Considering above discussion on R factors, using the information on actual behavior of steel plate shear walls in the laboratory and during earthquakes, in the following, an attempt is made by the author to develop and propose tentative R factors for steel plate shear walls. Code-writing bodies for inclusion can consider these tentative R-factors in seismic codes after peer review and receiving consensual approval of the structural engineering community.

Table 4.1 shows values of R, as well as other design parameters, for selective seismic-force-resisting systems relevant to shear walls. The values in the table are those given in a similar, but more extensive table in IBC-2000 (and by UBC-97).

Table 4.1. Design Coefficients and Factors for Basic Seismic-force-resisting Systems (The values in the table are those given by the IBC-2000)

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
Steel eccentrically braced frames, moment-resisting connections at columns away from links	8	2	4	NL	NL	160	160	100
Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	7	2	4	NL	NL	160	160	100
Special steel concentrically braced frames	6	2	4	NL	NL	160	160	100
Ordinary steel concentrically braced frames	5	2	4 ½	NL	NL	160	160	100
Special reinforced concrete shear walls	6	2	5	NL	NL	160	160	100
Composite eccentrically braced frames	8	2	4	NL	NL	160	160	100
Special composite reinforced concrete shear walls with steel elements	6	2.5	5	NL	NL	160	160	100
Special steel moment frames	8	3	5.5	NL	NL	NL	NL	NL
Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL	NL	NL
Dual system with special moment frames and steel eccentrically braced frames, moment-resisting connections, at columns away from links	8	2.5	4	NL	NL	NL	NL	NL
Dual system with special moment frames and steel eccentric braced frames, -moment-resisting connections, at columns away from links	8	2.5	4	NL	NL	NL	NL	NL
Dual system with special moment frames and special steel concentrically braced frames	8	2.5	6.5	NL	NL	NL	NL	NL
Dual system with special moment frames and special reinforced concrete shear walls	8	2.5	6.5	NL	NL	NL	NL	NL
Dual system with special moment frames and composite steel plate shear walls	8	2.5	6.5	NL	NL	NL	NL	NL

Notes: 1. This table only shows few systems and should not be used in actual design. For design refer to Table 1617.6 of IBC-2000.
2. NL=No Limit

Table 2 shows suggested values of R, the response modification factor, for some common types of steel shear walls. The test data indicate that if dual steel shear wall systems are designed such that the yielding of steel plate (in a tension field action) occurs before failure of other elements of the system, the wall will behave in a very ductile manner and can tolerate very large number of inelastic cycles without loosing either its seismic-force-resisting strength or gravity load carrying capacity (see Astaneh-Asl and Zhao, 2000). Therefore, to be consistent with R factors prescribed by codes for other systems, a value of R equal to 8.0 to 8.5 is proposed for special dual steel plate shear wall systems, see Table 4.2. The requirements of dual steel plate shear wall system are given in Table 4.2. However, it can be mentioned that the two requirements of such systems are: (a) the back-up moment frame should be a special moment frame and (b) the hierarchy of failure modes should be such that the yield failure modes occur prior to fracture failure modes in all elements that experience inelastic behavior and; (c) all connections are stronger than the connected members if members experience inelastic behavior.

Table 4.2. Proposed Design Coefficients and Factors for *Steel Shear Wall* Seismic-force-resisting systems
(The author A. Astaneh-Asl tentatively proposed the values in the table)

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

For un-stiffened steel plate shear walls that are not part of a dual system but are infill to a simply supported frame, a value of R factor equal to 6.5 is suggested. There is very limited number of tests done on steel shear walls inside simply supported frames. Intuitively, it is expected that the behavior of such system to be equal or better than the behavior of special concentrically braced frames for which an R-value of 6.5 is given in IBC-2000 (and UBC-97). For stiffened steel shear walls that are not part of a dual system, an R factor of 7.0 is suggested provided that b/t of stiffeners as well as plate panels is less than $52/\sqrt{F_y}$, the current limit for compact sections in seismic design (AISC, 1997).

4.2.c. Value of W_o***, for steel shear walls:*** The IBC-2000 (ICC, 2000) provides values of Ω_o, the over strength factor, for a variety of seismic-force-resisting systems. The factor is used to amplify seismic forces in design of specified structural elements and their connections to adjoining elements (SEAOC, 99). However, currently there are no values of Ω_o , for steel shear walls in US codes. Table 4.1 shows values of Ω_o and other design parameters for a number of typical systems given by IBC-2000 (and by UBC-97). By comparison to other systems and considering the results of available tests, a value of Ω_o equal to 2.5 is proposed for stiffened and un-stiffened shear walls as part of a dual system or standard system, see Table 4.2.

4.2.d. Value of C_d***, for steel shear walls:*** Suggested values of C_d, the deflection amplification factor, for a number of steel shear wall systems are given in Table 4.2. The selection of these values was based on being consistent with C_d values for systems that exhibit similar stiffness, strength, and post yield behavior and ductility. The C_d values for these similar systems are given in Table 4.1.

4.3. Seismic Provisions for Steel Plate Shear Walls

The previous section discussed issues related to the *Demand* side of the design equation: *Demand < Capacity and* how to establish earthquake loads for steel plate shear walls. This section discusses issues related to *Capacity* side of the design equation. These issues for seismic design of steel structures are currently addressed by "*Seismic Provisions for Structural Steel Buildings"(AISC, 1997), developed* and published by the American Institute of Steel Construction Inc. Although there are no specific provisions in this document on steel plate shear walls, there are many provisions for seismic design of other systems that are equally applicable to steel plate shear walls. In the following sections, these provisions are discussed and some suggestions are provided that, after being subjected to the professional review and refinements, can be incorporated into the seismic design codes.

4.4. AISC Seismic Provisions Directly Relevant to Steel Shear Wall Systems

The AISC Seismic Provisions for Structural Buildings (AISC, 1997) gives values of Ω_o, the over-strength factor for moment frames, eccentric braced frames and all other systems as 3, 2.5 and 2 respectively. Suggested values of Ω_o for steel shear walls are given in Table 4.2.

The AISC Seismic Provisions (AISC, 1997) has Expected Yield Strength, F_{ye}, defined by the following equation to be used in design of certain connections or related members. In chapter 5, when design recommendations for steel plate shear walls are discussed, in some cases, instead of specified yield stress, the Expected Yield Strength is used.

$$F_{ye} = R_y F_y \tag{4.7}$$

Where, F_{ye} is the specified minimum yield strength and R_y is a factor ranging from 1.1 to 1.5 depending on the grade of steel and weather the element is a rolled shape or a plate. The provisions given by AISC (1997) on Notch-toughness Steel (Section 6.3 of AISC, 97) equally applies to steel plate shear walls.

The provisions of AISC (1997) on Connections, Joints and Fasteners (Section 7 of the AISC-97) and on Columns (Section 8 of the AISC 97) are equally applicable to steel plate shear walls.

When a steel shear wall is combined with a moment frame to result in a "dual" system defined by the U.S. codes, the moment frames being "Special" should satisfy the requirements of Sections 9 of the AISC Seismic Provisions (AISC, 1997). Since data on performance of a dual system where steel shear wall is combined with Ordinary moment frame, Intermediate moment frame or Special Truss Moment Frame is almost non-existent, at this time and until more data becomes available such combination is not addressed in this report. This does not necessarily mean that such combinations could not be viable systems; only the user needs to establish its actual performance through realistic testing.

AISC Seismic Provisions (AISC, 1997) has provisions on Quality Assurance, which is equally applicable to steel plate shear walls. Reviewing the AISC Seismic Provisions, it is clear that there is an urgent need for a section in the document devoted to Steel Shear Walls similar to sections 11, 12, 13, 14 and 15, which are devoted to seismic design of various types of moment frames and braced frames.

4.5. Information on Shear Wall Design from Canadian Code

The National Building Code of Canada (CCBFC, 1995) has provisions on modeling and design of steel plate shear walls. The steel plate shear wall system covered in the Canadian code consists of relatively thin plate and without horizontal or vertical stiffeners. The story shears are assumed to be carried only by the tension field action of the thin plate after its buckling. The relatively small buckling capacity of the compression diagonal in the plate is ignored.

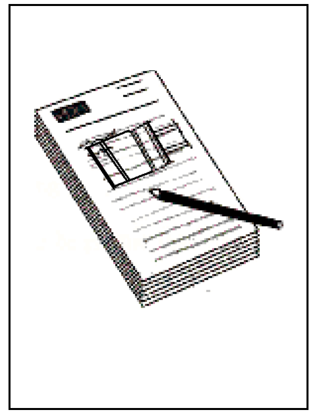
The Canadian Code divides steel plate shear walls into three categories of Ductile, Nominally Ductile and Ordinary. Table 4.3 provides the main characteristics of these three systems with associated R-factors. It should be mentioned that the R-factors used in the Canadian code are about 1/2 of the corresponding value in the U.S. codes. This is due to definition of terms in two codes and slight difference in values of parameters involved. For this reason, in the following table, the R-factor in the Canadian and the US codes are denoted as R_{CAN} and R_{US} respectively.

Table 4.3. Steel Plate Shear Wall Systems in Canadian Code and Their R-Factors

Type of Steel Plate Shear Wall	Requirement	R_{CAN} (Canadian)	R_{US} (US Equivalent)
Ductile	The frame containing the wall should be ductile moment frame	4.0	8.0
Nominally Ductile	The frame containing the wall should be nominally ductile moment frame (Intermediate in the U.S. definition)…..	3.0	6.0
Ordinary	No specific requirement for frame. It can be a frame with pin connections.	2.0	4.0

Appendix M of the National Building Code of Canada (CCBFC, 1995) is devoted to design requirement for steel plate shear walls. The Appendix states that the behavior of steel plate shear wall is similar to behavior of steel plate girders. The Appendix has provisions on how to replace a thin steel plate shear wall with diagonal pin-ended strips and what should be the angle of inclination of the strips. The Appendix has a provision regarding design of connections. It states that the connections of steel plate to the surrounding beams and columns should develop the steel plate.

5. SEISMIC DESIGN OF STEEL SHEAR WALLS



This chapter discusses seismic design and modeling of steel shear walls and provides seismic design recommendations.

5.1. Types of Steel Shear Wall Systems

Two types of steel shear walls are shown in Figure 5.1. These are *standard* and *dual* systems. In a standard steel shear wall system the beam-to-column joints have simple connections. As a result, the steel shear walls are assumed in design to be the only lateral-force-resisting element in the system. As mentioned in the previous chapter, it has been established (Liu and Astaneh-Asl, 2000) that the simple connections in steel structures have considerable moment capacity and behave in a "semi-rigid" manner rather than acting as a pin connection as the current practice assumes. In a dual steel shear wall system there are moment frames parallel to the plane of steel shear wall or in the plane of the shear wall. In this case, the moment frames act as back up systems to the primary lateral-force-resisting system which is the steel shear wall.

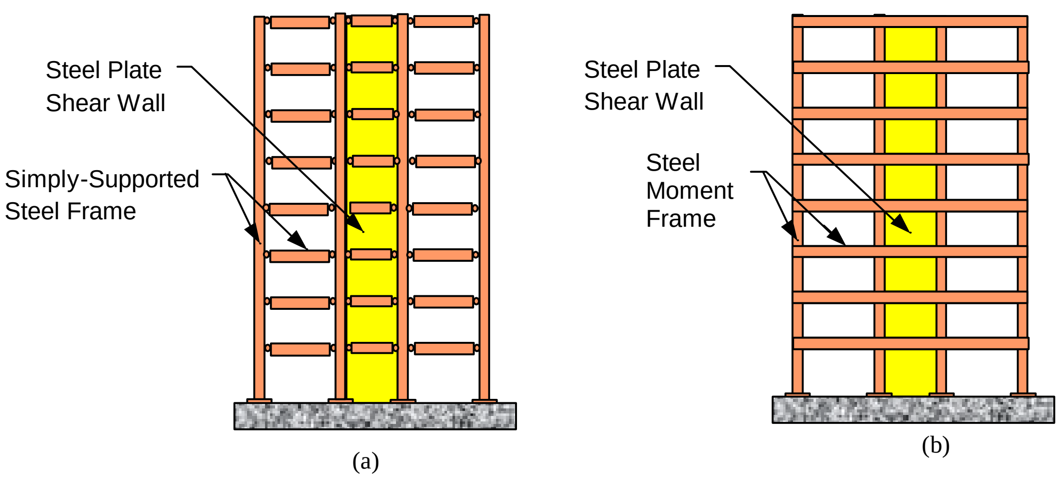


Figure 5.1 Two types of steel plate shear walls: (a) standard and; (b) dual system

Steel shear walls can be either stiffened or un-stiffened. In early applications of steel plate shear walls in the U.S., particularly for seismic retrofit, the walls were stiffened. Almost in all steel plate shear wall applications in Japan, according to existing literature, the steel plates are also stiffened. However during the last decade, a number of steel plate shear walls have been designed and constructed in the United States and Canada, using un-stiffened steel plate shear walls.

In using stiffened plate, the goal is to prevent buckling of the steel plate prior to shear yielding. In un-stiffened steel plate shear walls buckling of plate is permitted and the goal is to use diagonal tension field action in the wall to carry story shear. As a result, the stiffened steel shear walls tend to be thinner compared to un-stiffened steel plate shear walls when both are designed to carry the same shear.

In choosing stiffened or un-stiffened steel shear wall, the designer needs to consider seismic performance, architectural requirements, economy, ease of fabrication, transportation and erection. Seismic performance was discussed in the previous chapters. Both stiffened and un-stiffened steel plate shear walls, when designed properly, are expected to exhibit very desirable performance. From economical point of view, the un-stiffened shear walls are expected to be less costly since welding stiffeners to the steel plate is a labor-intensive activity. Of course, for the same thickness of steel plate, stiffened shear walls provide higher shear strength than the un-stiffened shear walls. Also in stiffened shear walls, the over-turning moment is shared by the boundary columns and the shear wall much like in an I-shape beam where the flange carries the bulk of bending effects and the web carries some bending moment and the bulk of shear.

In an un-stiffened wall the steel plate, due to initial buckling of compression diagonal, cannot participate in carrying significant amount of over-turning moment. As a result of additional compression in the boundary columns due to over-turning moments and the formation of tension field, the boundary columns in an un-stiffened shear wall may end up being somewhat heavier than the columns in a similar condition but with stiffened wall. However, the cost of welding stiffeners in today's fabrication shops, far exceeds the gain in strength of the wall and reduction in the forces imposed on the columns by stiffening the wall. Therefore, the author does not recommend common use of stiffened shear walls unless, due to openings being present and a need to stiffen the boundaries of these openings, the stiffened shear wall may be more economical.

5.2. Behavior of Steel Shear Walls Under Applied Shear

Figure 5.2 shows schematic variation of shear strength of a steel shear wall versus its slenderness ratio. The plot is actually based on the information established for plate girder behavior and design in the AISC Specification (AISC, 1999), but it is approximately applicable to the steel plate shear walls. Depending on slenderness of a shear wall, it can be categorized as Compact, Non-Compact and Slender as shown in Figure 5.2. Each gategory is discussed further in the following sections.

1. Category1, where the slenderness of the wall defined by h/t_w is less than λ_p equal to $1.10\sqrt{k_v E / F_{yw}}$. For definition of terms see Notations on Page iv of this report. Steel shear walls in this category are denoted as "compact". It is expected that under applied shear, as shown in Figure 5.3, the steel plate will yield in shear before buckling occurs. It is not economically feasible, nor necessary, to design un-stiffened steel plate shear walls to behave in a compact and plastic manner and have an h/t_w less than $\lambda_{p.}$ However, stiffened steel shear walls can be designed to develop this condition. This subject and how to design steel shear walls to achieve full plastic condition are discussed further later in this chapter.

2. Category 2, where the slenderness ratio h/t_w is larger than λ_p but smaller than λ_r equal given as $1.37\sqrt{k_v E / F_{yw}}$. This category is denoted "non-compact". It is expected that walls that fall in this category are expected to buckle while some shear yielding has already taken place. In this case, the story shear is resisted by the horizontal components of the tension and compression diagonal forces as shown in Figure 5.3.

3. Category 3, where the wall is very slender and its $h/\ t_w$ is greater than λ_r. Shear walls in this category can be called "Slender" and are expected to buckle while almost elastic.

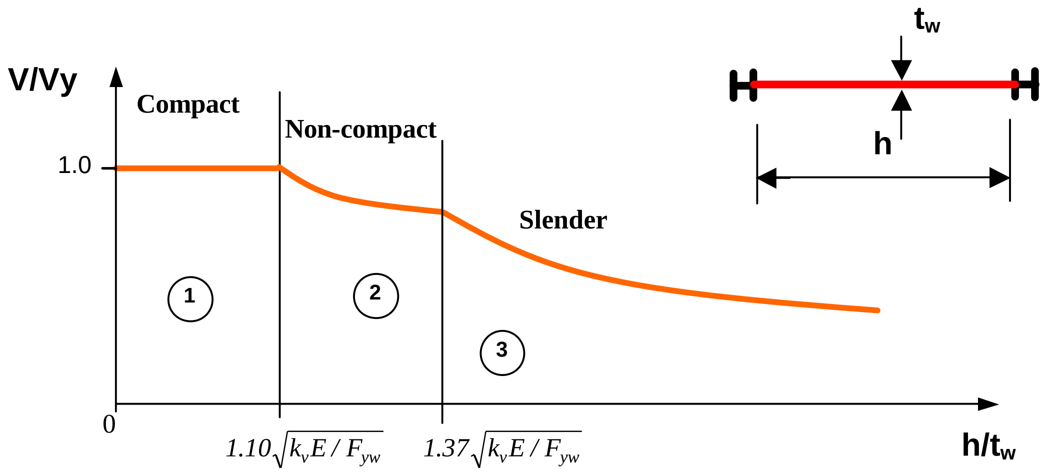


Figure 5.2. Three regions of behavior of steel shear walls

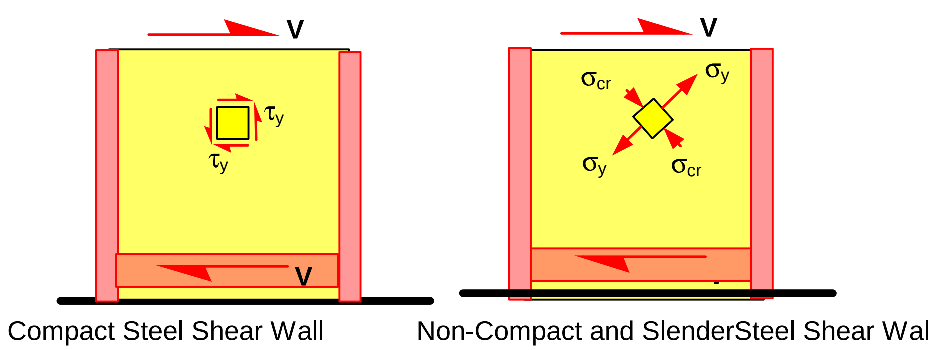


Figure 5.3. Steel shear walls resisting shear in "shear yielding" and tension field action

5.3. Design Criteria for Performance Based Design of Steel Shear Walls

When steel shear walls are designed using R, Ω_o and C_d values given in Chapter 4, the design criteria should be such that the system is sufficiently ductile and has enough over-strength. In order to achieve such performance with high ductility and over-strength in design, the following design procedure is developed and proposed. The basis of this procedure in general is to ensure that the ductile failure modes occur before brittle failure modes and inelasticity starts first in non-gravity carrying members of the system and then if necessary spreads into gravity load carrying elements towards the end of seismic event and in a controlled manner such that progressive collapse does not occur.

5.4. Developing Seismic Design Procedures for Steel Shear Wall Systems

The steps taken in developing seismic design procedures for steel plate shear walls are given below. These are the same steps taken by the author in developing design procedures for shear connections (Astaneh-Asl et al, 1989), bolted moment frames (Astaneh-Asl, 1995), column tree moment frames, (Astaneh-Asl, 1997) and gusset plates (Astaneh-Asl, 1998).

Steps taken in developing proposed design procedures:

1. An extensive literature survey is conducted to collect the information on the actual behavior of the system
2. Failure modes (limit states) of the system are identified
3. Failure modes are grouped into "ductile" and "brittle". The yield failure modes in general are considered ductile unless in rare occasions because of constraints on plastic flow, the yielding is not as ductile as desired. On the other hand the fracture failure modes are generally considered brittle. Buckling failure mode, depending on whether it is inelastic or elastic buckling, is considered ductile or brittle respectively. Slippage of bolts is considered ductile and the most desirable limit state for seismic design.
4. Failure modes are placed in a hierarchical order such that: (a) for members that can experience inelastic behavior, ductile failure modes should occur prior to brittle failure modes and; (b) non-gravity carrying elements, such as wall plate, reach their governing limit state prior to gravity carrying members do.
5. Design equations are developed for all failure modes such that the hierarchical order of the failure modes is materialized.

In the following the application of above steps to seismic design of steel shear walls is explained. The resulting proposed design procedures are given at the end of this chapter.

5.4.a. Major failure modes

The failure modes of typical steel plate shear walls are:

Failure modes of steel plate wall

1. Slippage of bolts (ductile).
2. Buckling of the steel plate (ductile).
3. Yielding of the steel plate (ductile).
4. Fracture of wall plate (brittle).
5. Fracture of the connections of steel wall to boundary columns and beams (brittle).

Failure modes of top and bottom beams

6. Shear yielding of top and bottom beams (ductile).
7. Plastic hinge formation in top and bottom beams (ductile).
8. Local buckling in the top and bottom beam flanges or web (ductile if $b/t \leq \lambda_p$).
9. Fracture of moment connections of the beams in dual systems (brittle).
10. Overall or lateral-torsional buckling of beams (brittle).
11. Fracture of shear connections of beams (brittle).

Failure modes of Boundary Columns

12. Plastic hinge formation at the top and bottom of columns (ductile).
13. Local buckling of boundary columns (ductile if $b/t \leq \lambda_p$).
14. Overall buckling of boundary columns (ductile if $\lambda_c = (KL/\pi r)\sqrt{(F_y / E)} \leq 1.0$.)
15. Tension fracture of boundary columns or their splices (brittle).
16. Yielding of base plates of boundary columns in uplift (ductile)
17. Fracture of anchor bolts or base plates at the base of columns in uplift (brittle)
18. Fracture of column base plates in bending and/or uplift (brittle)
19. Failure of foundations of the wall (brittle).

5.4.b. Hierarchical order of Failure modes

To obtain a desirable and ductile performance, the above failure modes can be listed with respect to their desirability. This hierarchical order of failure modes is shown in Figure 5.4. The hierarchical order is arranged such that the ductile failure modes of the wall itself, which is usually a non-gravity carrying element, occur first followed by ductile failure modes of the top and bottom beams and finally by ductile failure modes of the boundary columns. The brittle failure modes are generally arranged to occur after ductile failure modes. Again, among brittle modes also, it is desired that the brittle failure modes of the wall govern over those for the beams and columns.

Slippage of the wall boundary bolts or splices should not be considered a consequential failure mode. In fact, such slippage provides a mechanism of energy dissipation through friction and introduces some beneficial “semi-rigidity” to the structure. Of course the slippage should not occur under service lateral loads. Buckling of plate in slender shear walls does not appear to be detrimental in performance and have no significant effect on the ultimate shear strength and overall performance of the wall. If buckling of the wall, which will result in out-of-plane deformations, is creating serviceability problems, then stiffened shear walls should be used to delay buckling of the wall. Of course, as discussed earlier, such stiffened walls are expected to be more expensive. The yielding of diagonal tension field is the best mechanism of failure and should be established as the governing failure mode in seismic design. The fracture in tension or buckling in compression of boundary columns should be avoided in design since such failures can have serious stability consequences as well as very high cost of post earthquake repairs.

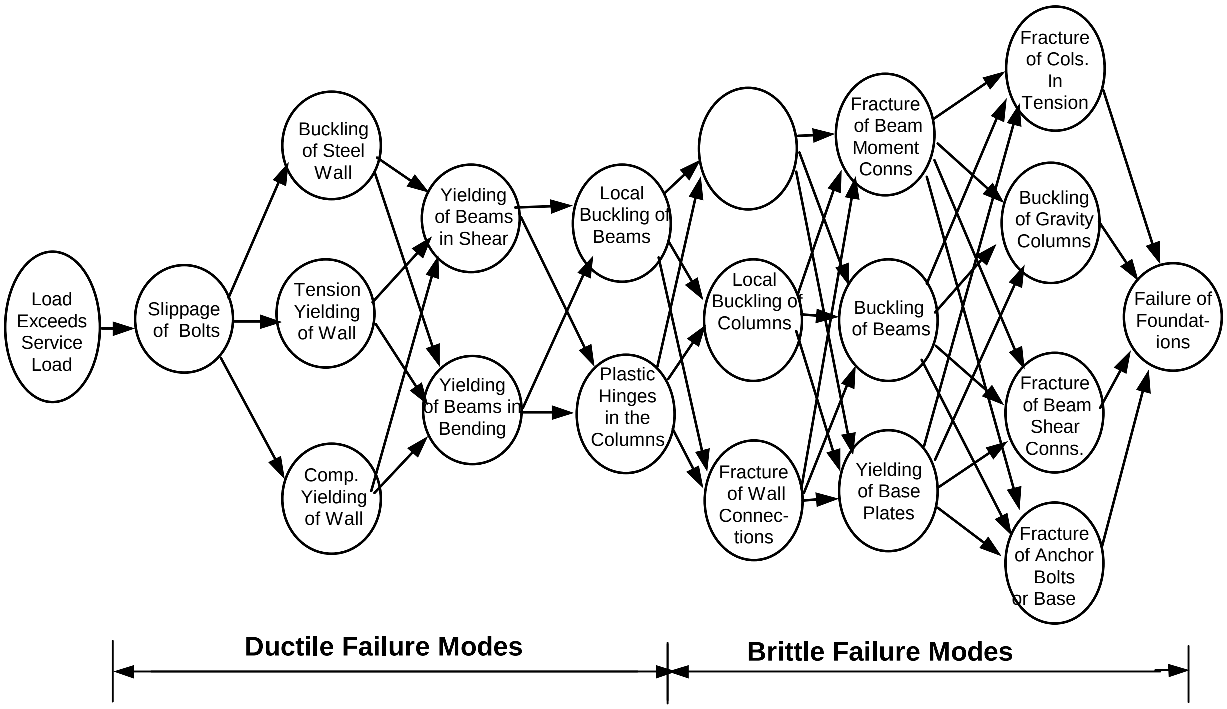


Figure 5.4. Major failure modes of typical steel plate shear walls
in the order of their desirability from left to right

5.4.c. Design Equations for Failure Modes

As was suggested earlier, to obtain a desirable seismic behavior, steel plate shear walls should be designed such that ductile failure modes, shown in Figure 5.4 above, are governing. To achieve this, the capacity calculated for the brittle failure modes is suggested to be greater than 1.20 times the capacity of the ductile failure modes. All failure modes listed in Figure 5.4 are well-known failure modes and are discussed in the AISC Manual of Steel Construction (AISC, 1994) and equations are provided that can be used to establish capacity of each failure mode.

5.5. Establishing Shear, Bending and Combined Shear-Bending Capacities of Shear Walls

Using the analogy of shear walls being similar to plate girders, the vast amount of design and modeling technology developed for plate girders can be adapted and applied to design of steel shear walls.

The main differences between plate girders and shear walls are:

1. Plate girders are seldom subjected to axial load. Although shear walls in many occasions are subjected to axial load due to gravity, such axial loads are usually resisted by the boundary columns and the steel shear wall itself is subjected to shear and bending only. The P-δ effects of axial load in the boundary columns on the overall stability of the shear wall system should be considered in the analysis.
2. The flanges in a plate girder are usually plates with relatively small bending stiffness in plane of the shear web. In a shear wall, the boundary columns act as flanges and since these columns are usually shapes instead of plates, they show larger bending stiffness and strength in plane of the wall than the flanges of a plate girder. The effect of bending stiffness and strength of the boundary columns is primarily in establishing the width and angle of inclination of the tension field.
3. The stiffeners in a plate girder nowadays are plates welded to one side of the web. In shear walls, the floor beams play the role of stiffeners. Obviously, the floor beams and if floor slab is attached to it acts as much stiffer and stronger stiffener than the plate stiffeners in the plate girder. The stiffness and strength of floor beams affects the boundary condition for buckling of plate. The floor beams in a shear wall system provided almost a fixed boundary for the steel plate. Also, the stiff and strong floor beams provide better anchor for the tension field particularly after the steel plate shear wall has yielded in floors above and below.
4. Plate girders are generally studied under monotonic loading or low-amplitude fatigue type loading while shear walls are expected to see relatively large inelastic cyclic loading. As discussed in Chapter 3, currently considerable data is available on cyclic behavior of steel plate shear walls.

Considering the above differences between steel plate girders and steel plate shear walls, it appears that none of the differences is major and none will result in non-conservative design. If anything, the plate girder equations might predict the capacity of the shear walls less than their actual capacity. Such under-estimation of capacity will result in somewhat conservative design. However, in design of foundations and other components that are designed to develop strength of the shear wall, one has to establish more realistic capacity for shear wall using more sophisticated model of behavior of shear walls. Valuable information on this can be found in SSRC Guide (SSRC, 1998) and papers by Elgaali and his research associates (see References).

5.5.a. Shear Capacity of Steel Shear Walls

Shear capacity of steel shear walls can be established using the following procedures which is an adaptation of procedures in the AISC Specifications (AISC, 1999) for steel plate girders. For the background on these equations, the reader is referred to SSRC Guide (SSRC, 1998) edited by Theodore V. Galambos.

The shear capacity of steel plate shear walls, in LRFD format, $f_v V_n$, where $f_v = 0.90$ and V_n is determined as follows:

A. For compact shear walls when $h/t_w \le 1.10\sqrt{k_v E / F_{yw}}$

$$V_n = 0.6 A_w F_{yw} \tag{5.1}$$

B. For non-compact and slender shear walls when $h/t_w > 1.10\sqrt{k_v E / F_{yw}}$

$$V_n = 0.6 A_w F_{yw} \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \tag{5.2}$$

Where k_v is given by:

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{5.3}$$

The value of k_v should be taken as 5.0 if a/h is greater than 3.0 or $[260/(h/t_w)]^2$. The value of C_v is given by AISC (1999) as:

(a) For $1.10\sqrt{\dfrac{k_v E}{F_{yw}}} \le \dfrac{h}{t_w} \le 1.37\sqrt{\dfrac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.10\sqrt{k_v E / F_{yw}}}{h/t_w} \tag{5.4}$$

(b) For $\dfrac{h}{t_w} > 1.37\sqrt{\dfrac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_{yw}} \tag{5.5}$$

A_w in the above equations is the shear area of the plate equal to $(d_w)(t_w)$ and V_n is the minimum shear capacity of the wall based on minimum specified yield strength. The above equations are adapted from plate girder equations given by the AISC Specifications (1999). In design, the following equation should be satisfied:

$$V \leq f\, V_n \tag{5.6}$$

where, V is the applied factor shear established by the analysis.

After design of steel plate, an "expected shear capacity", V_{ne}, should be calculated using the actual shear area of the wall and the expected yield strength of steel. In stiffened shear walls, if vertical stiffeners are substantial and particularly are made of channels, the cross sectional areas of the vertical stiffeners should also be included in this calculation. The expected shear strength of the wall is greater than the nominal shear strength given by above equations. The main reason is strain hardening and the fact that today, actual yield strength of steel is generally greater than the minimum specified value (i.e. 36 ksi for A36). The expected shear capacity of the wall will be used in design of other elements of the system such as connections and boundary beams and columns. V_{ne} is given by:

$$V_{ye} = C_{pr}\, R_y\, V_n \tag{5.7}$$

For definition of all terms in this report, please refer to Page iv. C_{pr} is a factor, originally introduced by FEMA-350 (FEMA, 2000) for moment frames and here it is used to increase the shear yield capacity of the steel plate due to strain hardening. The strain-hardened material is assumed to have a yield point equal to the average of F_y and F_u. Therefore, C_{pr} can be written as:

$$C_{pr} = (F_y + F_u)/(2F_y) = 1 + F_u/2F_y \tag{5.8}$$

R_y is a factor to account for uncertainty in the specified value of F_y and is given by AISC (AISC, 1997). According to AISC (1997), R_y for steel plate shear walls can be taken as 1.1.

5.5.b. Bending Capacity of Shear Wall

When Shear Wall is stiffened to reach shear yielding prior to buckling of plate, considerable percentage of over-turning moment can be resisted by the stiffened wall plate. However, in un-stiffened shear walls the bulk of over-turning moment is resisted by boundary columns.

5.5.c. Combined V-M and V-M-P Capacities of a Steel Shear Wall

For stiffened shear walls and un-stiffened but "compact" shear walls, since the wall is expected to participate in carrying shear as well as bending and axial force, an interaction curve should be used to relate V, M and P acting on the shear wall. Depending on the location of a panel in a stiffened shear wall, the panel will be subjected to a combination of shear and normal stresses acting on its edges. Since stiffened shear walls are expected to be quite ductile, significant redistribution of stress can take place prior to failure of the wall. Therefore, the capacity of the stiffened wall will be summation of capacities of its individual panels.

The SSRC Guide (SSRC, 1998) has a summary on this issue and provides following equation for the combined effects of shear and normal stresses acting on the edges of steel plate:

$$\frac{s}{s_{cr}} + \left(\frac{t}{t_{cr}}\right)^2 = 1.0 \qquad (5.9)$$

For un-stiffened and slender shear walls, since the bulk of axial load and overturning moment is resisted by columns, and the shear is carried by the tension field action of the wall, there seems to be no significant interaction of shear and axial force and moment that warrants use of an interaction equation.

5.6. Design of Connections of Steel Shear Wall Plates to Boundary Beams and Columns

Two typical details of connections of steel plate shear wall to boundary beams and columns are shown in Figure 5.5. The welded connections should be designed such that the connection plates (fin plates) and welds develop the “expected shear yield” strength of the wall given in previous section as $C_{pr} R_y V_n$

If field-bolted connections are used, the bolts should be slip critical and develop the “expected shear strength” of the wall. Even if bolts are slip critical, it is expected that during the cyclic loading of the wall, the bolts slip before the tension field yields. However, such slippage will occur at a load level considerably above the service load level and not only is not harmful but can be useful in improving the seismic behavior. Until more test results on bolted shear walls become available, it is strongly recommended that even if wind loads govern, the slip-critical bolts be used to connect the walls to boundary members and the bolts be designed not to slip under a load equal to or greater than 1.2 times the service wind load.

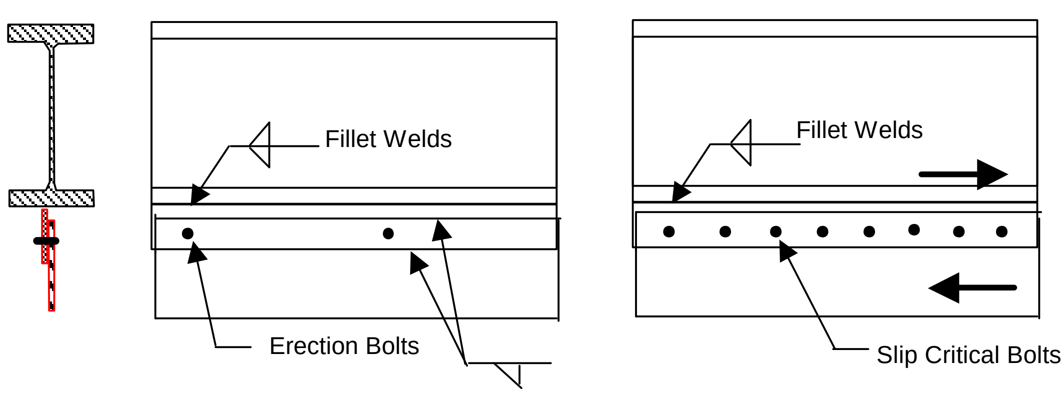


Figure 5.5. Connection of steel plate shear wall to boundary beams and columns

5.7. Design of top and bottom beams and columns

For dual shear wall systems when beams and columns are part of the special moment frames, provisions of special moment frames should apply to design of these beams and columns. For shear walls that are standard shear wall and not dual, the boundary beams and columns should be designed such that the governing failure mode is one of their ductile failure mode, as were listed earlier in this Chapter, and not the brittle failure modes To achieve this, one needs to check the brittle failure modes and ensure that their capacity is at least 1.2 times the capacity of ductile failure modes.

In addition, the boundary beams and columns of shear walls should satisfy the following b/t requirements given by the AISC Seismic Provisions (AISC, 1997):

$$b_f/2t_f \leq 52/\sqrt{F_y} \tag{5.10}$$

The above equation in non-dimensional form can be written as:

$$b_f/2t_f \leq 0.31/\sqrt{E/F_y} \tag{5.10a}$$

$$\mathrm{h_c/t_w} \leq 520/\sqrt{\mathrm{F_y}} \tag{5.11}$$

The above equation in non-dimensional form can be written as:

$$h_c/t_w \leq 3.10/\sqrt{E/F_y} \tag{5.11a}$$

For definition of terms please refer to Page iv. The SAC Joint Venture (SAC, 2000) suggests a limit of $418/\sqrt{F_y}$ for welded moment connections instead of $520/\sqrt{\mathrm{F_y}}$ given by the AISC (1997). The reason for choosing more relaxed limit of $520/\sqrt{\mathrm{F_y}}$ for web buckling of beams and columns in this system is due to the fact that in the shear wall systems discussed here, webs of columns and beams are part of the shear wall and it is unlikely that the webs will buckle prior to buckling of the wall. It is recommended that the web thickness of beams and columns in an un-stiffened steel shear system be at least the same thickness as the wall plate.

5.8. Modeling Steel Shear Walls in the Analysis

Depending on the steel plate shear wall being "compact", "non-compact" or "slender" and depending on the capabilities of the analysis software to handle plate buckling, shear walls can be modeled in several ways. In the following a number of modeling techniques for compact and non-compact/slender shear walls are briefly summarized. The three categories of shear walls; "compact", "non-compact" and "slender" can be seen in Figure 5.2.

A. Modeling of "Compact" Steel Plate Shear Walls:

In compact shear walls, the steel plate is expected to yield in shear before buckling. Therefore in the analysis, the compact shear walls can be modeled using full shell elements and isotropic material. It is suggested that the wall panel be modeled using at least 16 shell elements (4x4 mesh) per panel. The shear force V acting on the cross section of the wall can be calculated by adding up the shear in the elements. This applied shear force represents the demand on the wall and should be less than or equal to shear capacity of the wall established later in this chapter.

B. Modeling of "Non-Compact" Steel Plate Shear Walls:

The available test results on non-compact steel shear walls are mostly on shear walls with much higher h/t_w than $l_p = 1.10\sqrt{k_v E / F_{yw}}$. Most un-stiffened steel plate shear walls tested under cyclic loading had h/t_w greater than $l_e = 3.53\sqrt{k_v / F_{yw}}$. Until more cyclic testing is done on shear wall specimens with h/t_w between l_p and l_e, the procedure outlined in this section should be considered applicable only to un-stiffened shear walls with h/t_w greater than l_e.

In slender shear walls, steel plates are expected to buckle along compressive diagonals under relatively small shear forces. After buckling, the tension field action of the tension diagonal becomes the primary mechanism to resist shear force in the wall. This behavior should be considered in the analysis by modeling shear walls using shell elements that can buckle. If the analysis software does not have the capability to consider the buckling of shells, to simulate the buckling of compression diagonal the shear walls can be modeled using full shell elements and anisotropic material. Using anisotropic materials enables the analyst to assign different moduli of elasticity and shear moduli to three principal directions of the wall such that the compression diagonal will have much less stiffness and will attract much less shear in proportion to its buckling capacity than the tension diagonal. It is suggested that the wall panel be modeled using at least 16 shell elements (4x4 mesh) per panel. The shear force V acting on the cross section of the wall can be calculated by adding up the shear in the shell elements. This applied shear force represents the demand on the wall and should be less than or equal to shear capacity of the wall.

If the analysis software does not have anisotropic or at least orthotropic shell elements, one can replace the steel plates with a series of truss members (struts) along the tension field. Researchers in Canada have suggested two models for this purpose shown in Figure 5. 6. The reader is referred to the corresponding references on the figure for more information on these models.

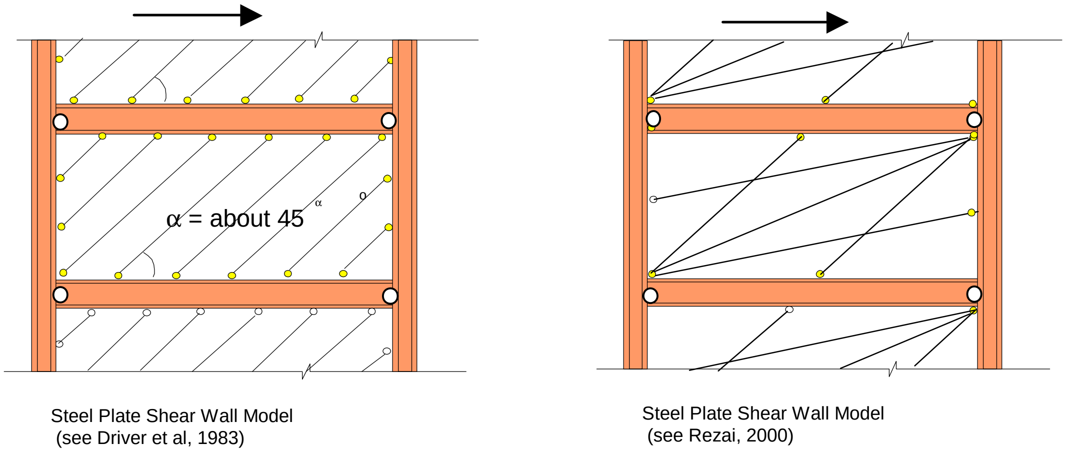


Figure 5.8. Two models proposed for replacing shear wall with truss members

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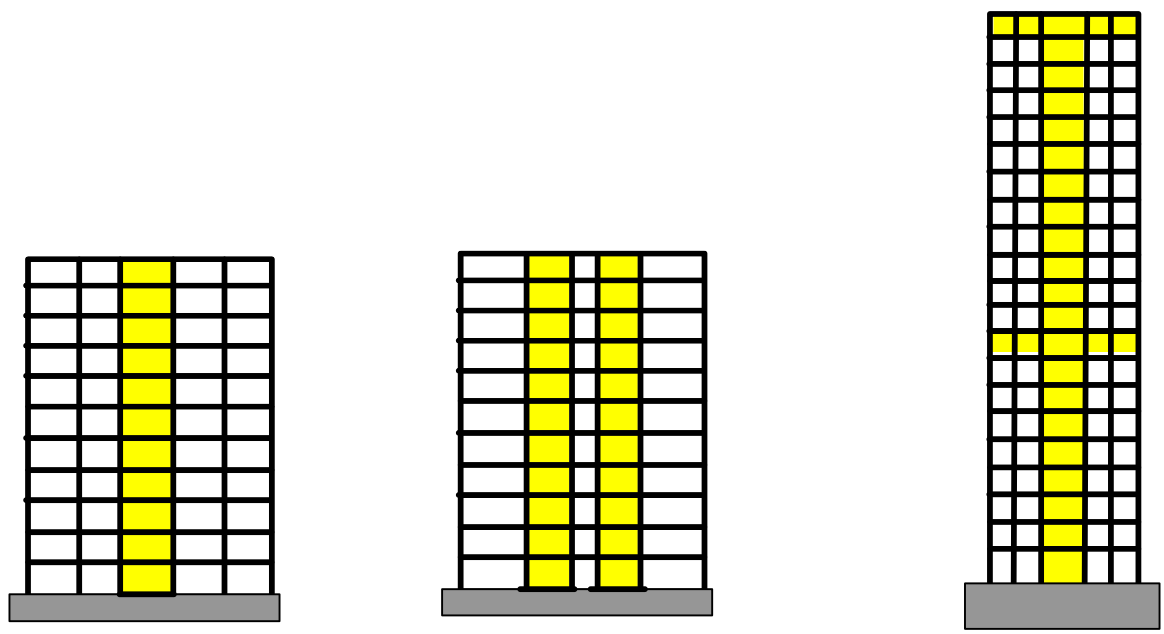
APPENDIX- SUGGESTED STEEL SHEAR WALL SYSTEMS AND DETAILS

Figure A.1. shows typical steel shear wall systems that structural engineers have used in the past or the author is suggesting to the reader as being economical and efficient systems. Figures A.2, A.3 and A.4 show three details for steel shear walls. The detail in Figure A.2. was used by Youssef (2000) and Troy and Richard (1988) in design of the 30-story high rise in Dallas and the 6-story hospital in Sylmar near Los Angeles. Chapter 2 of this report included a summary of these two structures. The system shown in Figure A.3 is a typical steel plate shear wall system with the steel plate either field bolted or field-welded to the columns and girders. The system shown in Figure A.4 is completely field bolted system where one or two-story assemblies of steel plate, boundary columns and boundary beams are fabricated in the shop. Then shop-welded pre-fabricated segments are erected in the field using bolted or welded splices in the columns and steel walls.



Standard Shear Wall **Coupled Shear Walls** **Out-rigger Shear Wall**

Figure A.1. Typical configurations for steel plate shear wall systems

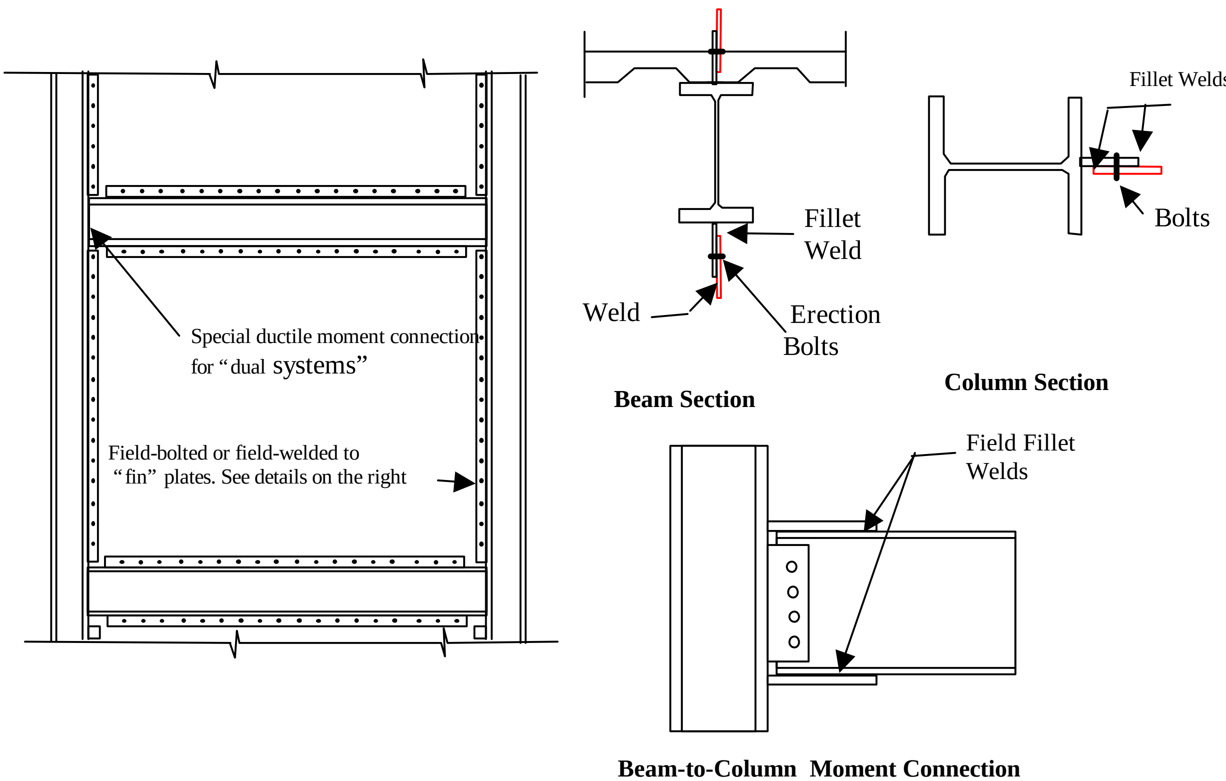


Figure A.2. A Suggested steel plate shear wall system

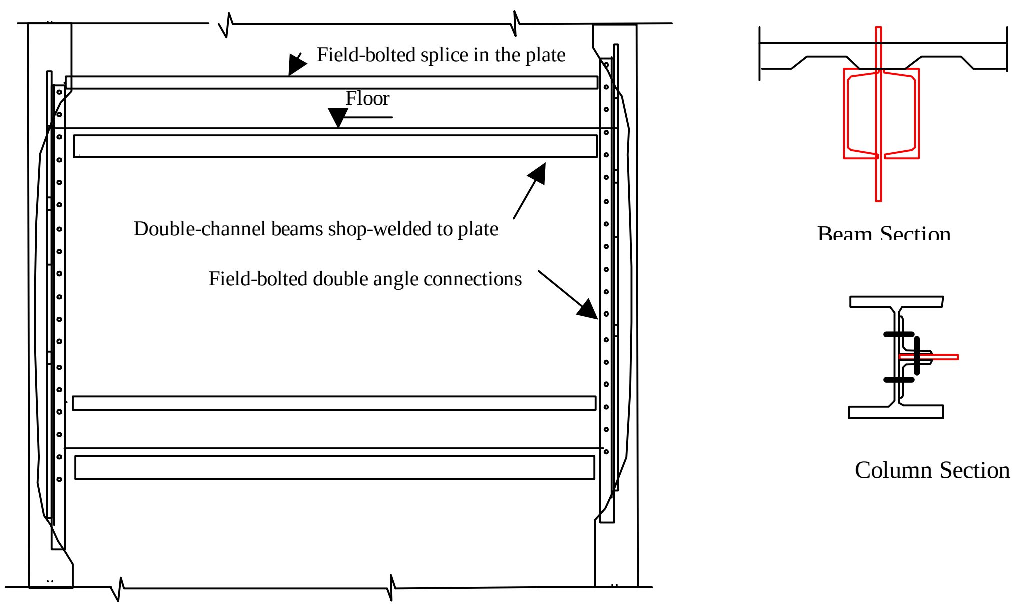


Figure A.3. A shop-welded, field-bolted steel plate shear wall system
(Adopted from an application by Troy and Richard, 1988)

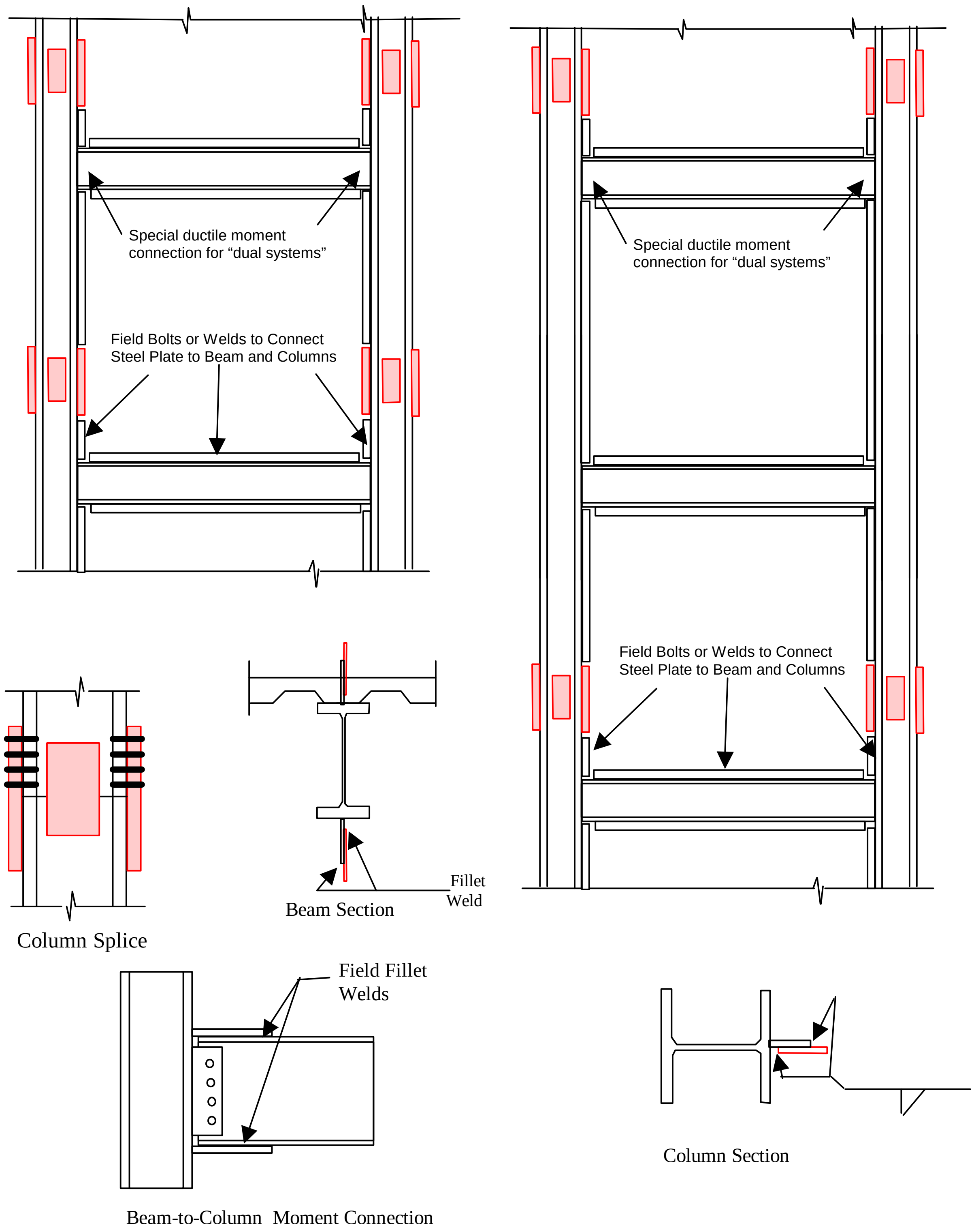
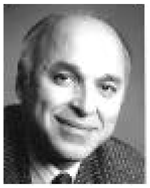


Figure A.4. Shop-welded, field-bolted steel plate shear wall

About the author….



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