13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2576

CYCLIC BEHAVIOR OF AN INNOVATIVE STEEL SHEAR WALL SYSTEM

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SUMMARY

Steel shear walls have been used more frequently in recent years as the lateral load-resisting system in the design and retrofit of high-rise buildings. This paper concentrates on the experimental studies of an innovative steel shear wall system used in buildings in USA and presents a summary of test results.

Steel plate shear wall system studied herein consists of steel plate shear walls placed inside a multi-bay steel moment frame. In this system, steel walls are welded to the boundary steel moment frame. The steel moment frame consists of very large concrete filled steel tubes (CFT) at the edges, internal wide flange (WF) columns, and horizontal WF beams. Most of the gravity is resisted by the CFT columns, and lateral loads are resisted by the dual system consisting of the moment frame and the steel shear wall.

Cyclic static tests were conducted on two half-scale specimens representing this system with different span-to-height ratio for the steel wall panels. Both specimens showed highly ductile and stable inelastic behavior, in the sense that they were able to tolerate more than 30 cycles of inelastic shear displacements before reaching an inter-story drift more than 0.03. Inter-story drift herein is defined as lateral movement of the floor divided by the story height. Throughout the test, the gravity load carrying CFT's remained essentially elastic while non-gravity carrying lateral load resisting elements underwent well-distributed and desirable yielding. The experimental results and their implication in seismic design are summarized and discussed.

INTRODUCTION

Reinforced concrete shear walls have been widely used as lateral load resisting system in concrete buildings in the past, especially in high-rise buildings. In steel buildings, in most cases concrete shear walls are used with a perimeter steel moment frame to resist seismic effects. However, tension cracks and compression crushing failures in a concrete shear wall can result in spalling and splitting failure of the wall, and lead to serious deterioration of stiffness and reduction of energy-dissipation capacity. Furthermore, the casting and curing of concrete wall makes the construction not so efficient compared to

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other systems such as braced frames or moment frames. In recent years, more attention has been paid to steel shear walls that could be constructed economically and efficiently in high rise buildings. Seismic behavior of this system and the design guidelines for them are therefore of high interest.

Steel plate shear walls have been used as the primary lateral load resisting system in several modern and important structures in Japan and USA during the last 20 years. In Japan, stiffened steel plate shear walls were used in new building construction since the 1970's, and recently research has been conducted on steel shear walls made of low strength steel and composite shear walls with "grooves" . (Astaneh-Asl [4]) In USA, stiffened shear walls were first used in seismic retrofit of existing hospitals in California. Stiffened shear walls and shear walls with "grooves" are seldom studied and used in USA due to the high labor cost. The focus of industry and this paper is on un-stiffened steel shear walls that were proven to be more efficient in USA.

BACKGROUND

Steel Shear Wall Systems and an Innovative System

Three common types of steel plate shear wall systems are shown in Figure 1. In type I, the steel plate shear wall is welded (or bolted) to the boundary elements in only one bay. The system is a "dual" system with the moment frame and the steel shear wall in the same bay. In type II, two type I systems are connected by the horizontal coupling beams and work together. Type III is an innovative steel shear wall system developed and used by Magnusson Klemencic Associates. It is similar to type II, except that the edge columns in the moment frame are very large concrete-filled steel tubes (CFT's) instead of the usual wide flange (WF) section. Due to the high axial stiffness of the CFT's, most of the gravity load is carried by them. These large CFT columns are connected to horizontal beams using special moment connections to form special ductile moment frame.

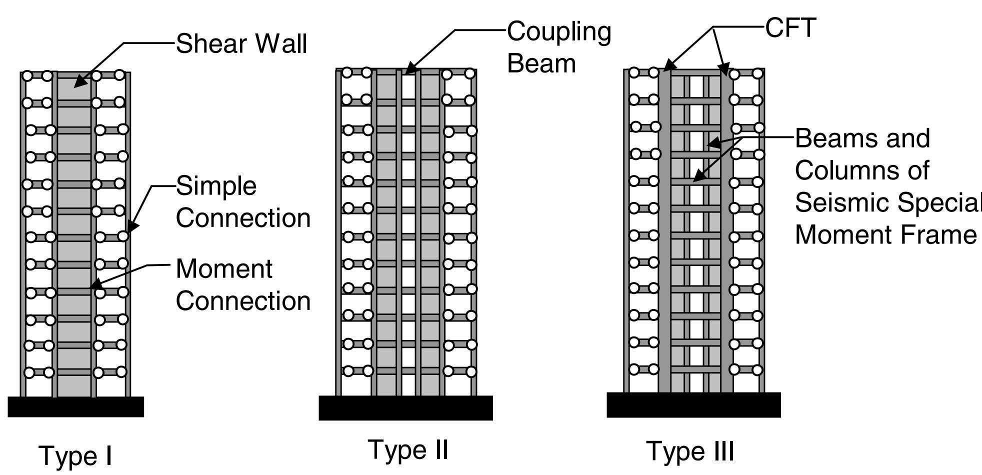


Figure 1. Three common types of steel shear wall systems. (Zhao [3])

The focus of steel shear wall project was the behavior of the innovative steel shear wall system Type III, which was developed and used by Magnusson Klemencic Associates. The gravity load resisting system of the building consisted of WF columns at perimeter and four CFT columns at the core. The lateral load resisting system consisted of a steel shear wall system in one direction and steel braced frame system in the other, as shown in Figure 2.

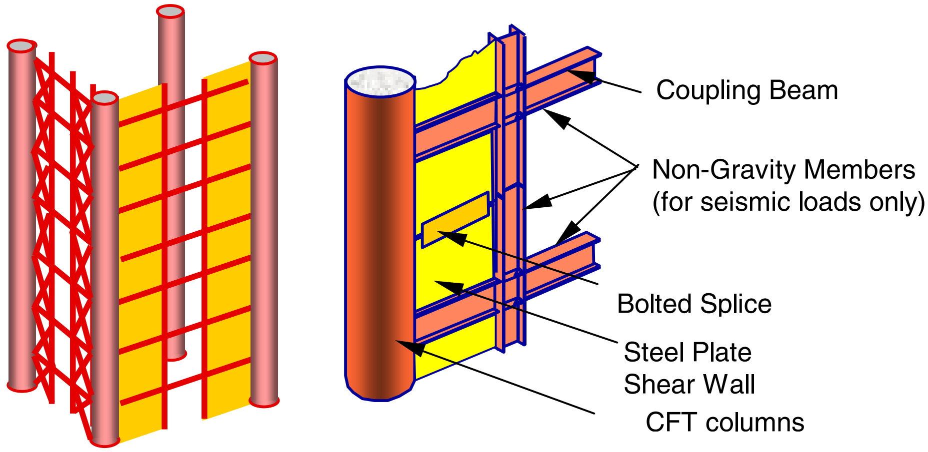


Figure 2. Lateral Load Resisting System of a Steel Building

Objectives of the Study

The main objectives of the research program summarized herein were to:

1. Collect information on actual behavior of steel shear walls and results of cyclic tests conducted on the system;
2. Conduct analytical parametric studies on the innovative steel shear wall systems to identify key parameters affecting seismic behavior;
3. Conduct cyclic tests on the system and establish its cyclic behavior regarding strength, stiffness, energy dissipation and damageability characteristics;
4. Develop design guidelines and recommendations.

CYCLIC TEST ON STEEL SHEAR WALL SYSTEM

Specimens

The test program consisted of cyclic testing of two half-scale specimens. The specimens were constructed as sub-assemblies of the prototype building over two floors (for Specimen One) and three floors (for Specimen Two) with different span-to-height ratio for the walls, as shown in Figure 3. The test specimens represented a dual lateral load-resisting system where steel shear wall is the “Primary” lateral load resisting system welded to a “Back-up” system of special moment-resisting frame. Due to symmetry, specimens only included half of the system it represented in the actual building and a roller was put at the end of the coupling beam to simulate the boundary conditions, as shown in Figure 3.

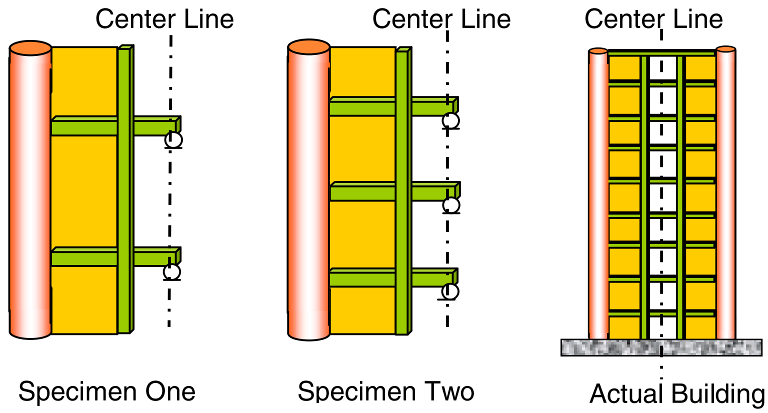


Figure 3. Specimens as Sub-assemblies of Actual Building

The specimens were composed of a boundary CFT column, WF beams, interior WF column and steel plate walls, as shown in Table 1 and Figure 4. The steel tube, WF column and beams were made of A572 Grade 50 with specified yield stress of 345 MPa (50 ksi), and the wall plate was made of A36 with specified yield stress of 248 MPa (36 ksi). The concrete had a minimum specified f'c of 21 MPa (3 ksi). The WF beams and columns were welded to each other and welded to the steel plate shear wall in the shop using flux cored arc welds with E70T-7 electrodes and a specified strength of 483MPa (70 ksi). Then the steel unit is welded to the CFT column in the field. The two specimens were fabricated according to the specifications and details provided by the designers (Magnusson Klemencic Associates).

Table 1. Components of Test Specimens

Specimen No. & Designation	Steel Wall Plate Thickness	CFT column		Beam Section*	Column Section*
		Thickness	Dia.		
1 two-story	6 mm(1/4 inch)	8 mm (5/16 inch)	610 mm (24 inch)	W18x86	W18x86
2 three-story	10 mm(3/8 inch)	8 mm (5/16 inch)	610 mm (24 inch)	W18x86	W18x86

* Properties of cross sections refer to the AISC Manuals (AISC [1]).

The WF beams were continuous through the WF column with their top and bottom flanges welded to the column using full-penetration welds. The coupling beams were the extension of WF beams. Two sliding load cells were added to the end of each coupling beam to simulate the roller condition. The ends of the coupling beams were reinforced to support the load cells, as shown in Figure 4.

Each shear wall had an all-bolted splice at mid-height of story. The bolts in the steel plate splices were 16 mm (5/8 inch) diameter A490 slip-critical bolts tightened according to the AISC Specifications (AISC [1]). The splice plates were 6 mm (¼ inch) thick A36 steel plates on both sides. The WF columns in both specimens had also all-bolted field splice at mid-height of each story. All bolts were 22 mm (7/8 inch) diameter A490 Slip Critical bolts tightened as per AISC Specifications (AISC [1]). The material of plates used in splices was all specified as A36 steel.

The CFT column was a 610 mm (24 inch) diameter steel tube with a thickness of 8 mm (5/16 inch) filled with concrete. The concrete-filled tubes in the actual structures carry a large portion of the gravity load. To represent the gravity compression, eight 38 mm (1-1/2 inch) diameter DYWIDAG W/FPU 1034 MPa (150 ksi) were placed inside the tube before casting concrete. Besides the DYWIDAG bars, there were also shear studs welded inside the steel tube to ensure the composite action. Prior to fabrication of specimens, the tube was cut in half, shear studs were welded and then the two halves of tubes were welded back to each other using 5 mm (3/16 inch) partial penetration welds to make the full tube. The shear studs in specimens were 19 mm (¾ inch) diameter by 76 mm (3 inch) length WHSS Nelson studs. The studs were spaced in a staggered pattern with 165 mm (6.5 inch) spacing in circumferential direction and 254 mm (10 inch) in longitudinal direction making the actual spacing of each stud from nearby studs almost 254 mm (10 inch).

The moment connections of WF beams to CFT column consisted of eight 19mm (¾ inch) diameter steel deformed re-bars (four on each flange) which were embedded in the concrete-filled steel tube and fillet-welded to the flanges of the WF beams, as shown in Figure 5.

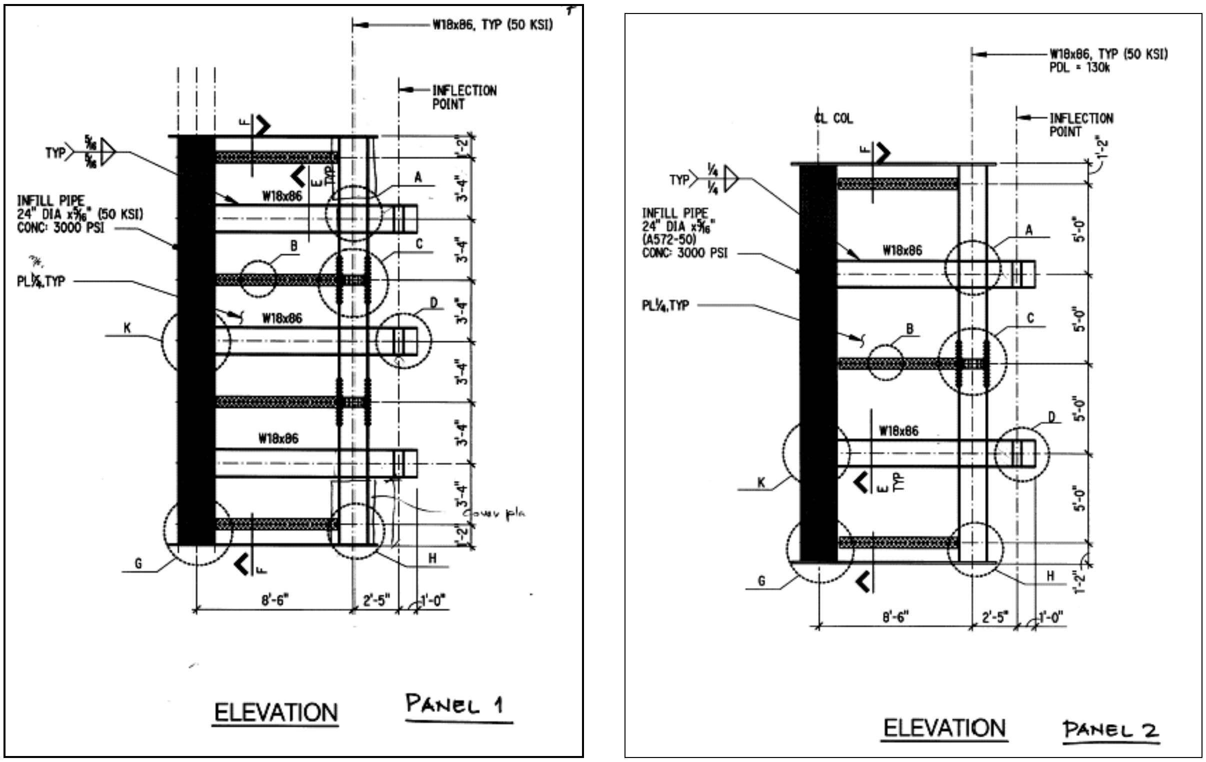


Figure 4. Structural Details of Test Specimens

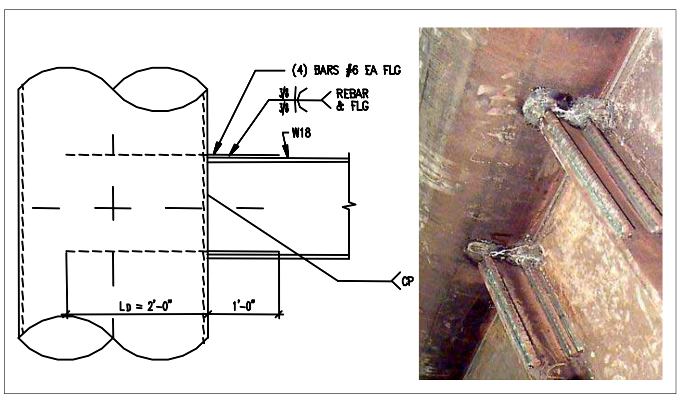


Figure 5. Details of Wide Flange Beam to CFT Column Connections

Test Set-up

The test set-up is shown in Figure 6. Main components of the test set-up are: Actuator, Top Loading Beam, Bottom Reaction Beam, R/C Reaction Blocks, Bearing Support and Bracings.

The Actuator, Top Loading Beam, Bottom Reaction Beam and R/C Reaction Blocks were the same as the components of the test set-up for the composite shear wall project (Zhao [5]). Cyclic shear forces were applied by the only actuator to the top of the specimen, therefore all the stories in the specimen had the same story shear, which represented the shear distribution in the prototype high-rise building.

In order to simulate bracing effects provided to the shear wall system by the story floors in actual buildings, two types of bracings were used in the specimen. A point bracing was used on the top Loading beam to brace the middle of this beam and prevent out of plane movement. A set of braces was also used to brace the end of coupling beams and to prevent out of plane movements. The bearing support was designed to support the ends of coupling beams with load cells and transfer the reactions to laboratory floor.

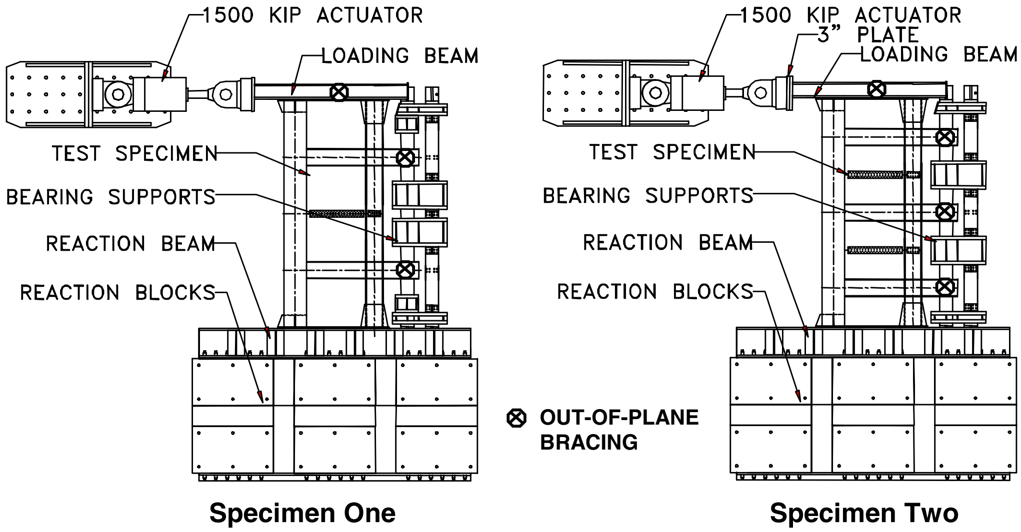


Figure 6. Test Set-up and Components

Loading History

After 1994 Northridge earthquake, the SAC Joint Venture developed a loading history and test procedures. The loading history developed by SAC Joint Venture and included in the Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens (SAC [2]) was later included in the AISC Seismic Provisions (AISC [1]).

Due to differences between a beam-column connection and steel shear wall system, the SAC protocol could not be used directly. However, in developing loading history an attempt was made to be consistent with the SAC Protocol. The specimens were tested under cyclic displacement regime. Cyclic displacements applied to both specimens were the same as shown in Figure 7. The overall drift is defined as actuator displacement divided by the total height of the specimen 6.19m (20 ft - 4 inch).

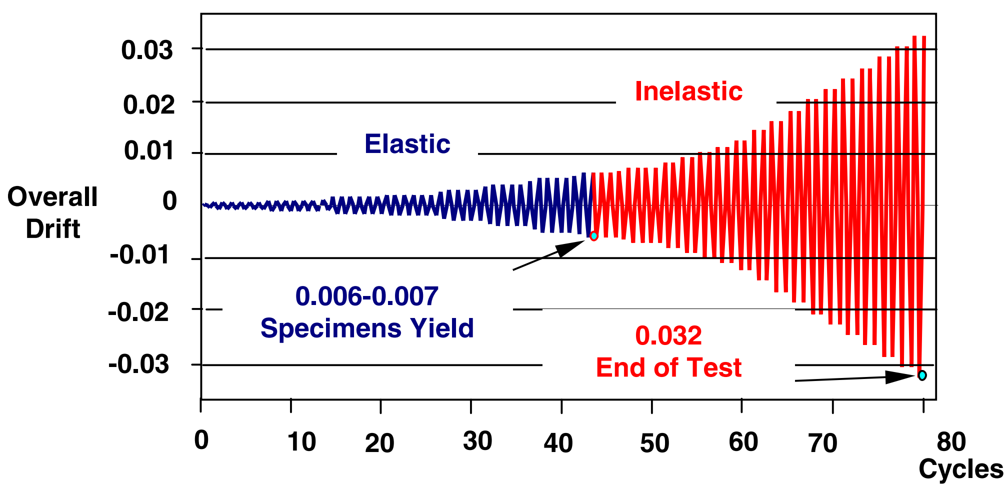


Figure 7. Loading History Applied to Both Specimens

Instrumentation

A number of Linear Variable Displacement Transducers (LVDT's) were installed on the specimens to measure the global as well as local displacement of critical points such as intersection points of member centerlines, the corner points of steel panels, the corner points of panel zones, etc. In this way movement of the members were monitored and important data such as panel zone deformation, shear wall panel deformation, slippage in the bolted splices and movement of the top and bottom flanges of a beam in a moment connection were measured. For safety purposes, a series of displacement transducers were mounted on the test set-up to detect any slippage of the actuator or the reaction blocks.

In order to measure strains at various critical locations, linear as well as "rosette" strain gauges were mounted on the specimen. Critical locations included quarter points on a cross section of the steel tube, flange and web of the column, middle point of the steel panel, quarter point of the steel panel, etc.

In order to measure reaction forces at the end of the coupling beams special load cells were designed and installed. The load cells consisted of solid steel cylinders with semi-circular ends to act as rollers. The cylinder portion was strain gauged and calibrated prior to tests.

CYCLIC BEHAVIOR OF STEEL SHEAR WALL SPECIMENS

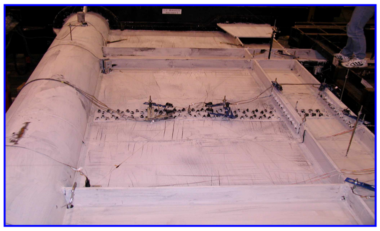
Steel Shear Wall Specimen One with Two Stories

Specimen One behaved in a very ductile and desirable manner. Up to overall drift of about 0.006, 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 overall drift of about 0.022, 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 of which 35 cycles were inelastic, before reaching an inter-story drift of 0.032 and maximum shear strength of about 4079 kN (917 kips). Notice here the inter-story drift value equals to the lateral displacement of a floor divided by the story height, which is different from the overall drift value used in loading history. At this drift level, 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 below 75% of the maximum capacity. The specimen was then considered failed. Behavior of Specimen One during cyclic testing is summarized in Table 2. Figure 8 shows Specimen One at various stages of cyclic loading.

Table 2. Key Test Observations on Behavior of Specimen One

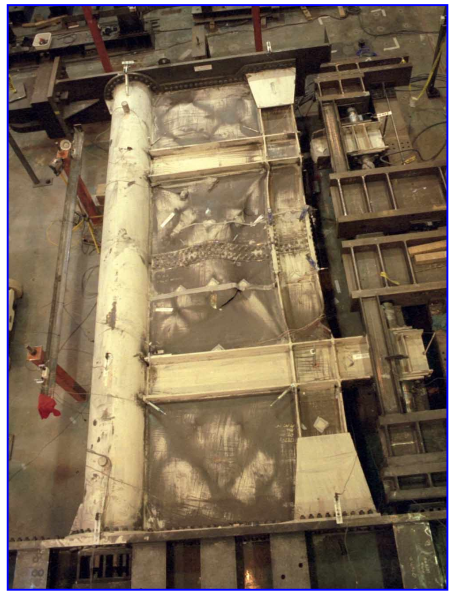
Loading Groups	Actuator Drift (Rad)	Description
0	0.0002	Very small warm-up cycles were applied to check test set-up & Instruments.
1	0.00075	Actual test started. Specimen remained elastic.
2	0.001	Specimen remained elastic.
3	0.0015	Bolts on reaction blocks made noise. Specimen still appeared elastic.
4	0.002	Big bang noise from reaction beam. Specimen remained elastic.
5	0.0025	Continuous bang and squeaking noise. Specimen remained elastic.
6	0.003	Bang and squeaking noise from time to time. Specimen remained elastic.
7	0.0035	Bang and squeaking noise from time to time. Specimen remained elastic.
8	0.004	Some minor non-linearity was observed on the force-disp. curves.

9	0.0045	Bolt slipped on base plate connection. No sign of non-linearity on the curves.
10	0.005	Hysteretic curves were getting slightly fatter, sign of non-linearity.
11	0.0055	Middle panel buckled in a second mode shape. Tension field formed. No major yielding. "**Proportional Limit Point**".
12	0.006	Entire middle panel yielded. Buckling mode changed. Force-disp. curve was still almost elastic. Slight yielding at column base. "**Significant Yield Point**".
13	0.0065	Yielding in the middle panel continued. Buckling changed to first mode with compression diagonal buckling from lower beam to upper beam.
14	0.007	Dropping of load at peak point indicated plate buckling. Column splice slipped slightly. Yielding at top beam to CFT column connection.
15	0.008	Permanent out-of-plane deformation of middle panel.
16	0.009	Yielding was visible in areas around panel zone. Yielding of top flange of bottom beam occurred between rebar to CFT column connections.
17	0.01	First and second buckling modes interacted in the middle panel. Two kinks formed in the right half of the panel.
18	0.011	More yielding occurred on steel shear wall panels and ends of WF column.
19	0.012	Fracture and more kinks in the middle panel. Clear bending of column.
20	0.014	Second fracture in the middle panel. More yielding at the end of top coupling beam and panel zones. Web of top beam yielded near the CFT column.
21	0.016	Local buckling at the base of WF column above the bottom beam. Epoxy fracture was observed showing yielding of rebar inside the CFT column.
22	0.018	WF column had permanent deformation and yielded heavily inside middle panel. Two more fractures and big out-of-plane bump formed in the middle panel.
23	0.02	Very obvious yielding at the WF column base inside the middle panel. Slight yielding of WF column around the base cover plate.
24	0.022	Plastic hinge formed and flange local buckled at the top coupling beam – WF column connection. WF column flange yielded at the ends inside middle panel.
25	0.024	Rebar's epoxy out at the connections of top and bottom beams to CFT column. Twisting at the locally buckled part of WF column above the bottom panel zone.
26	0.026	WF column flange locally buckled below the top beam panel zone. Rebar fractured at the bottom beam to CFT column connection. Corner of middle panel fractured in diagonal direction.
27	0.028	Top coupling beam fractured at the face of WF column. WF column base fractured at locally buckled area. "**Point of Maximum Shear Strength**".
28	0.030	Flange of WF column fractured below the top beam. Fracture of the WF column base inside the middle panel propagated into the already buckled web. The top coupling beam was totally separated from the horizontal WF beam. Load dropped to about 60% of maximum. "**Point of Maximum Ductility**".
29	0.032	Rebar yielded at the connection of top beam to CFT column. Small fracture occurred at "kink" locations of the top steel plate panel. Test stopped.



Specimen One Yielding

Top Coupling Beam



Specimen One at End of

Figure 8. Specimen One at Different Stages of Cyclic Test

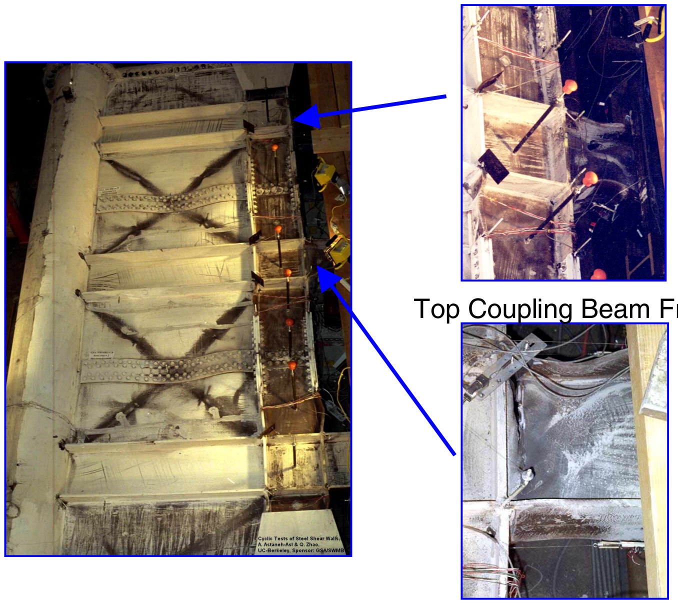
Steel Shear Wall Specimen Two with Three Stories

Specimen Two also behaved in a ductile and desirable manner. Up to overall drift of about 0.007, the specimen was almost elastic. At this drift level some yield lines appeared on the wall plate and the force-displacement curve started to deviate from the straight elastic line. During later cycles a distinct X-shaped yield line was visible on the steel plate shear walls. The specimen could tolerate 79 cycles, out of which 30 cycles were inelastic. The specimen reached maximum shear force of 5449 kN (1,225 kips) under an overall drift of 0.022. In the next drift level, the top (fourth story) coupling beam fractured at the face of the column (due to low-cycle fatigue). At an overall drift of 0.032, the CFT column fractured at the base and the load dropped below 75% of maximum strength, then the specimen was considered failed and the test stopped. Behavior of Specimen Two is summarized in Table 3. Figure 9 shows Specimen Two at end of cyclic loading.

Table 3. Key Test Observations on Behavior of Specimen Two

Loading Groups	Actuator Drift (Rad)	Description
0	0.0002	Very small warm-up cycle was applied to check test set-up & Instruments
0	0.0004	Very small warm-up cycle was applied to check test set-up & Instruments
1	0.00075	Actual test started. Specimen was elastic.
2	0.001	Specimen remained elastic.
3	0.0015	Specimen remained elastic.
4	0.002	Specimen remained elastic.
5	0.0025	Noise of friction. Specimen remained elastic.
6	0.003	Specimen remained elastic.

7	0.0035	Specimen remained elastic.
8	0.004	Specimen was still elastic.
9	0.0045	Specimen was still elastic.
10	0.005	Loud squeaking noise from bracing and load cell. Specimen remained elastic.
11	0.0055	Big bang noise. Specimen remained elastic.
12	0.006	Very minor vertical yield lines in members.
13	0.0065	Top,middle coupling beam flange and web yielded. "**Proportional Limit Point**".
14	0.007	The second story panel formed a tension field and diagonal yield lines were observed across the splices. "**Significant Yield Point**".
15	0.008	More yielding in the top and middle coupling beam, and third story wall panel.
16	0.009	Some tension field action shown in third story panel.
17	0.01	In the third story panel both shear yielding mode and buckling mode were clearly observed. In the second story panel buckling mode was more obvious.
18	0.011	WF column in the second story showed much yielding.
19	0.012	Very obvious X shaped tension field was formed in the middle wall panels. Some yielding in the middle horizontal beam web.
20	0.014	Very obvious buckling of two middle wall panels and distortion of wall splices. Some yielding in the bottom horizontal beam.
21	0.016	Kinks formed in the lower half of the middle wall panels.Change of vibration mode from first to second. WF column and middle panel zone heavily yielded.
22	0.018	Rebars at bottom and middle beam to CFT column connections had been elongated and epoxy was pulled out.
23	0.02	X shaped valley was formed on the middle panels. Kinks formed at quarter points of the valley. Flange and web of top coupling beam buckled slightly.
24	0.022	Obvious buckling in the first floor wall panel. Middle coupling beam had web and flange buckling near panel zone. Very slight web buckling at the WF column base in the second story. "**Point of Maximum Shear Strength**".
25	0.024	Top coupling beam fractured at the face of WF column, extending from top flange into web. Middle coupling beam has flange and web local buckling near panel zone. WF Column of second story had flange local buckling at the ends.
26	0.026	Base of the column showed slight flange local buckling. Wall splice in the third story slipped. More yielding of splice in the WF column on second story.
27	0.028	Top coupling beam totally separated. Kinks developed into fracture in the wall. Very obvious web and flange local buckling at the WF column base on the second and first story. Heavy yielding on the top horizontal beam.
28	0.030	CFT column fractured at the base and buckled.
29	0.032	First story WF column flange totally yielded and locally buckled. Load dropped to about 75% of maximum. "**Point of Maximum Ductility**". Test stopped.



Top Coupling Beam Fracture

Specimen Two at End

Middle Coupling Beam Fracture

Figure 9. Specimen Two at End of Cyclic Test

DISCUSSION OF TEST RESULTS

Both specimens behaved in a very ductile manner and could tolerate large number of inelastic cycles of shear displacements. Also, the behavior of shear wall system was very similar to behavior of steel plate girders subjected to shear.

Up to overall drift value of about 0.006 behaviors of both specimens were essentially elastic. After this overall drift value the steel shear walls buckled along the compression diagonal and developed tension field action along the tension diagonal. Both specimens were able to reach cyclic overall drift of at least 0.03 after undergoing 79 cycles, over 30 of the cycles being inelastic. Specimen One failed at the over drift of 0.03, while Specimen Two failed at the overall drift of 0.032, by losing more than 20% of their total shear strength. In both specimens failure was initiated by the fracture of the top coupling beam along column face and the total separation of the coupling beam from rest of the specimen. The hysteresis loops for the walls in both tests are shown in Figure 10. In the figure shear force is plotted against inter-story drift defined as the lateral displacement "Δ" of a floor divided by the story height "h" as shown in the figure. The maximum inter-story drift was over 0.05 for the wall panel in Specimen One and over 0.03 for the wall panels in Specimen Two.

In both specimens, the CFT column remained essentially elastic with limited yielding shown in later cycles. This indicates there's lower possibility of progressive collapsing under extreme seismic events. The non-gravity carrying members like steel plate, WF beams and WF columns had significant yielding and formed plastic hinges at the connections, which indicated effective energy dissipation during seismic events. The moment connections of horizontal beams to CFT column also behaved in a highly ductile manner.

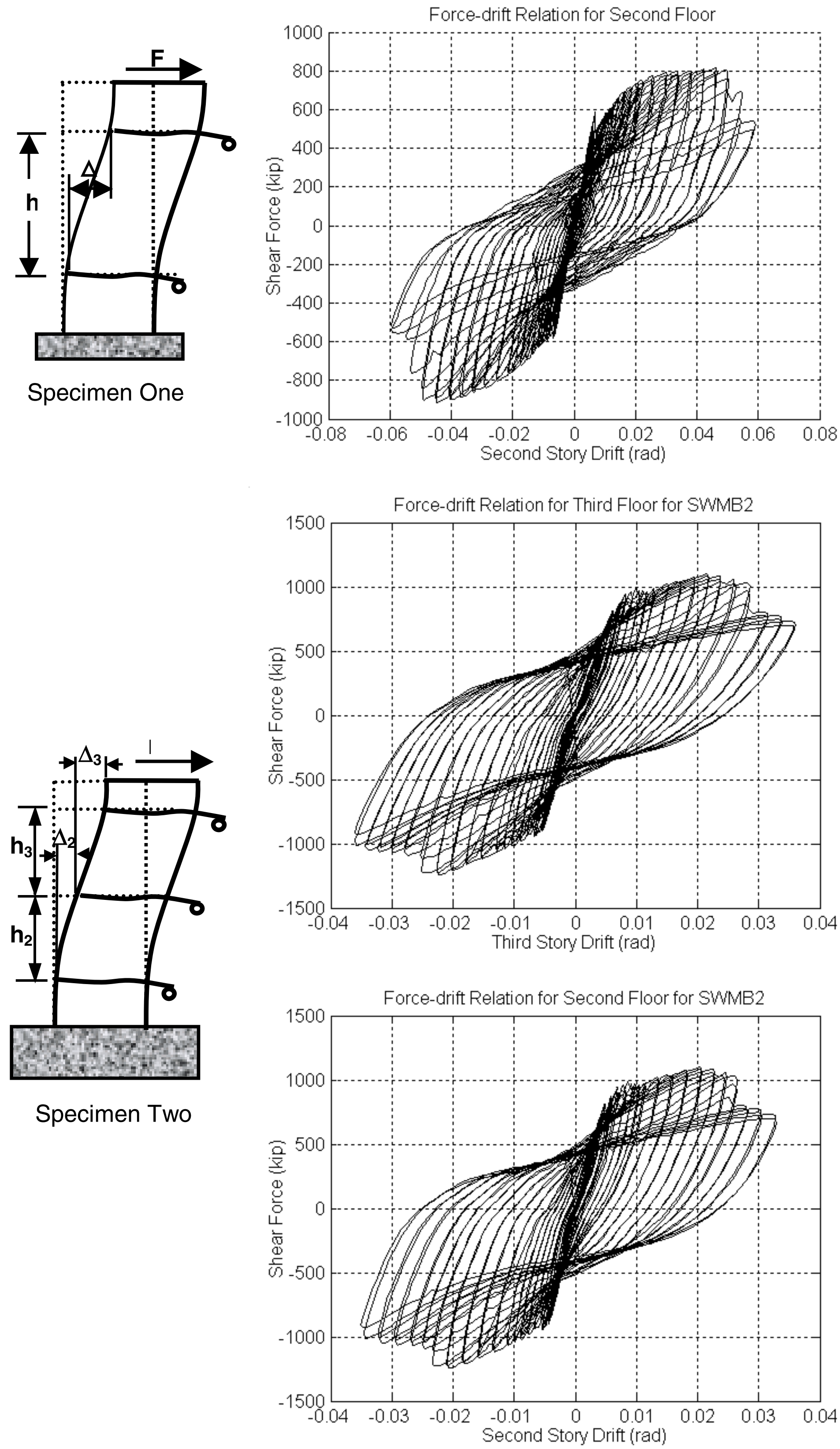


Figure 10. Shear Force vs. Inter-story Drift for Both Specimens

The coupling beams in both specimens developed plastic hinges at the face of columns and underwent large cyclic inelastic rotations, as shown in Figure 11. Eventually the top coupling beam fractured completely across the section at the plastic hinge location due to low cycle fatigue.

The bolted splices at mid-height of steel shear walls and the WF columns performed well and did not fracture. But there were several bolt slippages after the specimen yielded.

The steel plates in both specimens developed tension field action effectively. There were “X” shaped yield lines shown clearly on the plates. The plates also buckled diagonally when under compression.

In general, the overall behavior of the system was very ductile with the main gravity load carrying member (the CFT column) remaining essentially elastic and seismic components (shear wall, WF columns and horizontal beams) yielding and dissipating energy. This behavior made this type of steel shear wall an effective seismic-resistant system.

CONCLUSIONS AND APPLICATION IN SEISMIC DESIGN

The following conclusions were reached based on cyclic test results of shear wall specimens. The conclusions can be applied in seismic design of structures.

1. The behavior of shear wall system was very similar to behavior of steel plate girders subjected to shear. After yielding at overall drift of 0.006, the steel shear walls buckled along the compression diagonal and developed tension field along the tension diagonal.

2. The steel shear wall system was extremely ductile, exceeding inter-story drift of 0.03.

3. The gravity carrying element - CFT column, has remained almost totally elastic.

4. All lateral load resisting elements - steel plate, WF beams and WF columns yielded extensively and dissipated energy.

5. Coupling beams underwent large cyclic inelastic rotations near columns face. Both specimens failed after fracture of top coupling beam due to low cycle fatigue.

6. The bolted splices at mid-height of steel shear walls and internal wide flange columns performed well and did not fracture. However, during later cycles of testing, the bolted splices of columns were slipping.

7. Finally, observing the ease of fabrication and erection, the system appears to be a very efficient and economical system due to the fact that it is mostly shop-welded and field-bolted with only minimal field-welding with fillet welds that require minimal quality control.

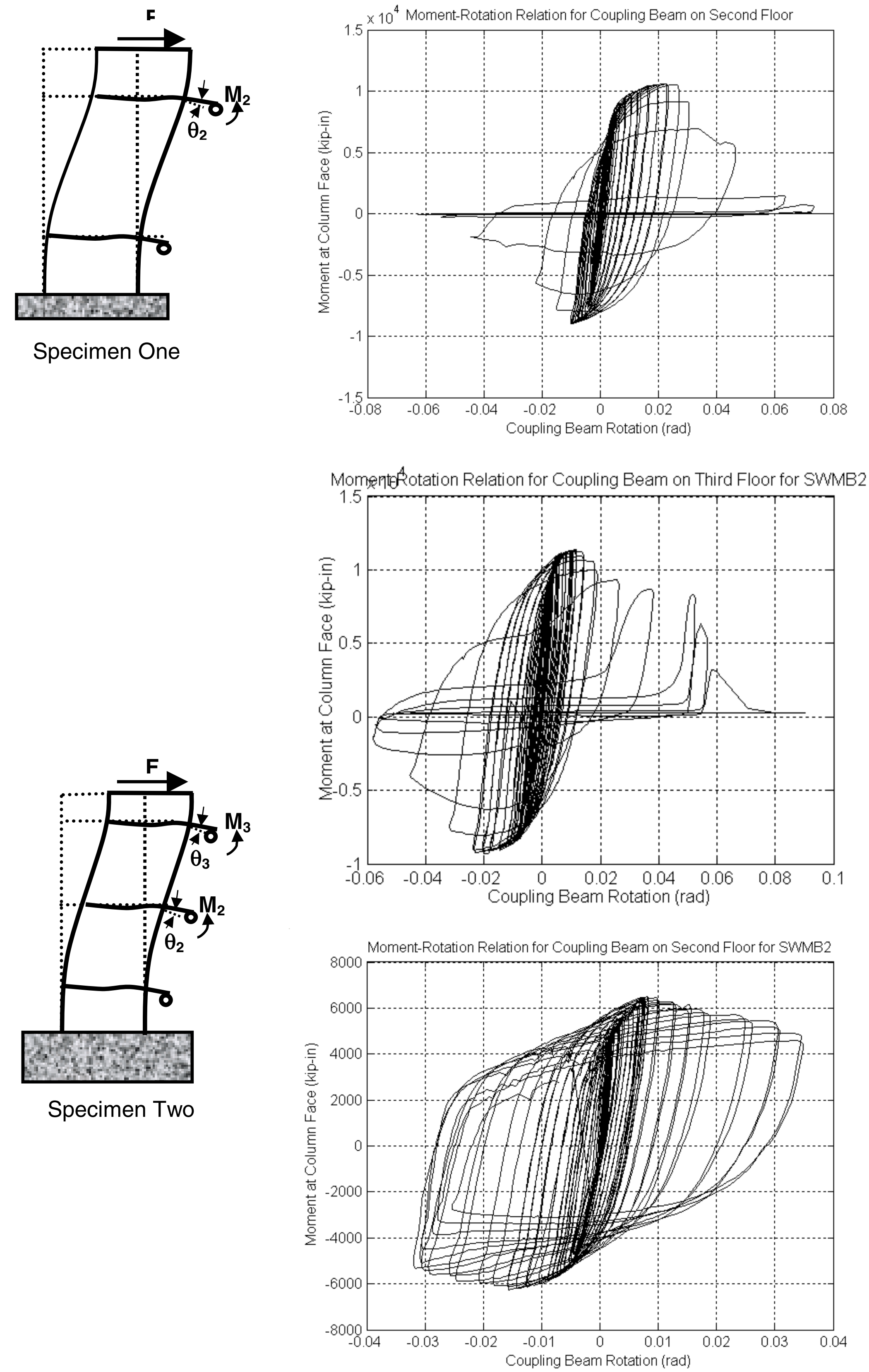


Figure 11. Hysteresis Behavior of Coupling Beams for Both Specimens

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