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Seismic Behavior of Steel Shear Wall Systems and Application of Smart Structures Technology

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Abstract

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. This paper also discusses potential application of smart structures technology into the design of steel shear wall system. The steel shear wall system studied herein consists of steel plate shear walls welded inside a multi-bay steel moment frame. The steel moment frame consists of large concrete filled steel tubes (CFT) at the edges, internal wide flange (WF) columns, and horizontal WF beams. Most of the gravity load is resisted by the CFT columns, and lateral loads are resisted by the dual system consisting of the moment frame and the steel plate shear wall.

Keywords: Smart Structures Technology, Steel Shear Wall, Seismic Engineering, Cyclic Test

1. Introduction

Shear walls have been widely used as lateral load resisting system in the past, especially in high-rise buildings. However, there were some concerns about using concrete shear walls in a steel high-rise building in high seismic zone, such as development of tension cracks and localized compressive crush under large seismic displacement, high base shear induced by relatively high lateral stiffness, high weight to strength ratio of concrete material, relatively long construction period compared to the steel parts due to casting and curing of concrete material, etc. Therefore, in recent years more and more attention has been paid to steel shear walls that could be constructed economically and efficiently in steel high-rise buildings.

During the last 20 years, steel plate shear walls have been used as the primary lateral load resisting system in several modern and important structures in Japan and USA. 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 steel shear walls with "grooves" (Astaneh-Asl, 2001). In USA, stiffened steel 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 is on un-stiffened steel shear walls that were proven to be more efficient in USA, and this paper will concentrate on a project conducted at the University of California, Berkeley to investigate the seismic behavior of an innovative steel shear wall system through large scale cyclic tests.

2. Project Background

The steel shear wall project described in the paper concentrated on the seismic behavior of the innovative steel shear wall system shown in Fig. 1, which was developed by Magnusson Klemencic Associates and used in one of their steel buildings. The system is a "dual" lateral load resisting system as defined in current codes

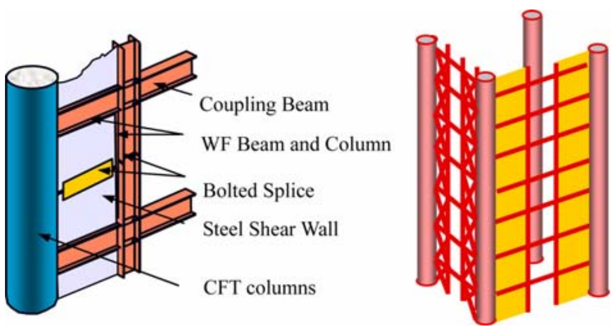

Figure 1. Main components of steel plate shear wall system.

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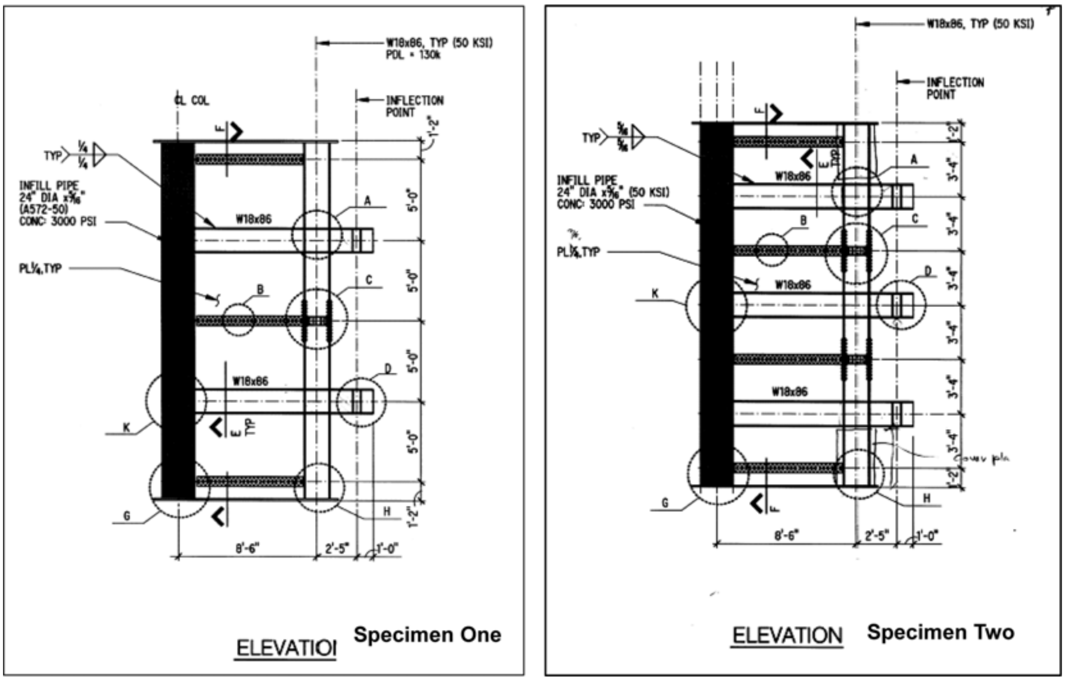

Figure 2. Structural details of test specimens.

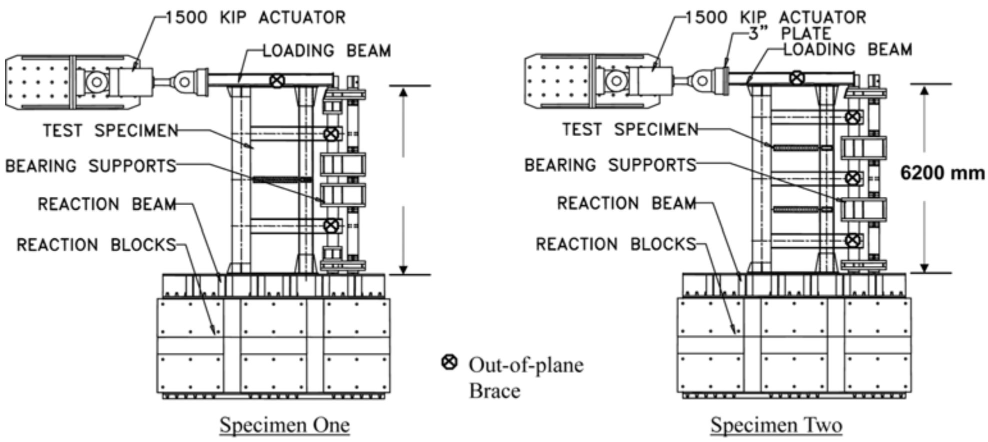

Figure 3. Steel shear wall specimens and test set-up.

(ICBO, 1997) with the steel shear wall (primary system) welded inside a multi-bay moment frame (secondary system) in a single bay. The edge columns in the moment frame are large concrete-filled steel tubes (CFT), which carry most of the gravity load due to high axial stiffness. In the core of the actual building, two bays of the steel shear wall systems are connected together by the horizontal coupling beams, as shown in Fig. 1.

3. Experimental Studies

3.1. Cyclic test on steel shear wall system

Two half-scale specimens were constructed as sub-assemblies of the prototype building over two floors (for Specimen One) and three floors (for Specimen Two) with different wall span-to-height ratio, as shown on Fig. 2. Each specimen included one or two full stories in the middle and two half-stories at the top and bottom.

The structural components are shown in Table 1. The steel tube, wide flange (WF) column and beams were made of A572 Gr. 50 steel with yield stress of 345 MPa, and the wall plate was made of A36 steel with yield stress of 248 MPa. The concrete had a minimum f'c of 21 MPa. Details of the test specimens are also shown in Fig. 2.

Test set-up for the steel shear wall tests is shown in Fig. 3. During the test, cyclic shear displacements were applied by the actuator to the top of the specimen through the top loading beam, and the shear force was transferred to the lab floor by the bottom reaction beam, reaction blocks and bearing support. Same cyclic displacements were applied to both specimens, which were established according to the SAC Protocol (SAC, 1997) as shown in Fig. 4. The overall drift in the figure is defined as the lateral displacement of the actuator divided by the total

Table 1. Components of steel shear wall specimens

Specimen No.	Steel wall thickness	CFT column		Beam section*	Column section*
		Tube thickness	Tube diameter		
Specimen one	6 mm	8 mm	610 mm	W18 × 86	W18 × 86
Specimen two	10 mm	8 mm	610 mm	W18 × 86	W18 × 86

*Cross section properties refer to the AISC Manual (AISC, 1994).

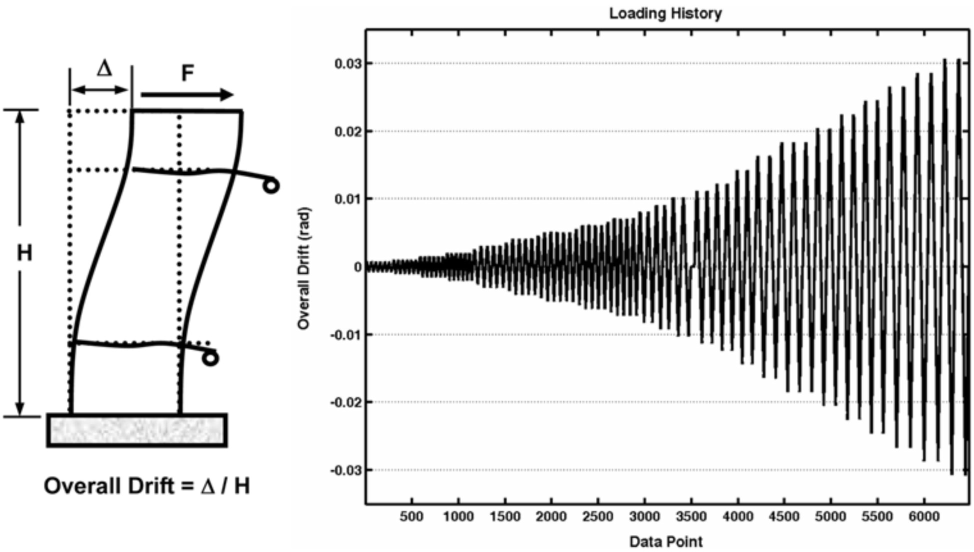

Figure 4. Loading history of steel shear wall tests.

height of specimens. A set of linear variable displacement transducers (LVDT) and strain gauges were installed on the test specimens and test set-up in order to measure the displacement and strain at critical locations of the specimen and monitor slippage of the test set-up.

3.2. Cyclic behavior of steel shear wall specimens

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. Here the inter-story drift value equals to the lateral displacement of a floor divided by the story height. 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.

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 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.

Both specimens behaved in a very ductile manner and could tolerate large number of inelastic cycles of shear displacements. Also, behavior of the shear wall system was very similar to behavior of steel plate girders subjected to shear. 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 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 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. Figure 5 shows specimens after the test, as well as the hysteresis curves for the stories in both specimens.

In both specimens, the CFT column remained essentially

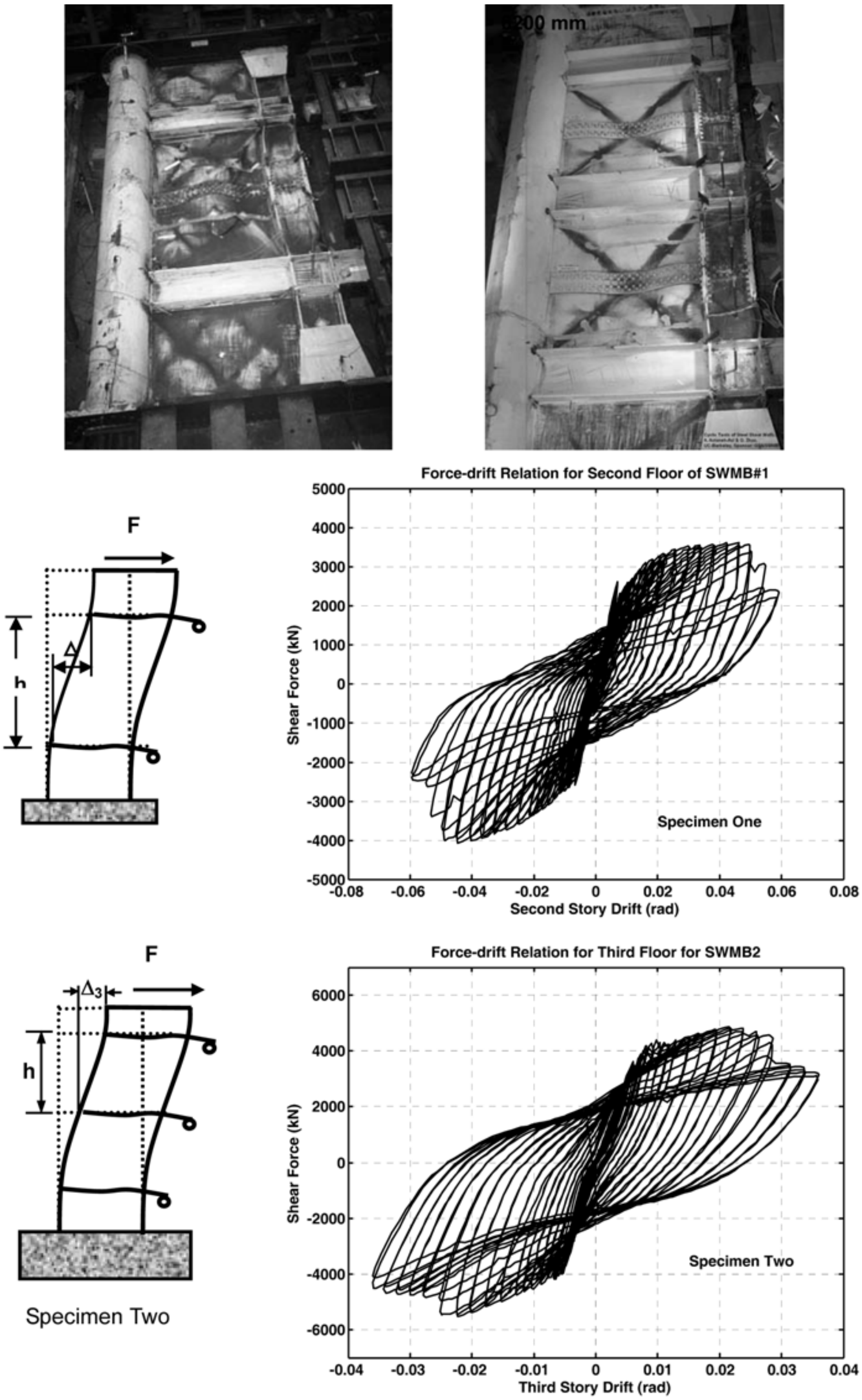

Figure 5. Steel shear wall specimens after the test and hysteresis curves for the stories.

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 such as 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.

The coupling beams in both specimens developed plastic hinges at the face of columns and underwent large cyclic inelastic rotations, as shown in Fig. 6. 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.

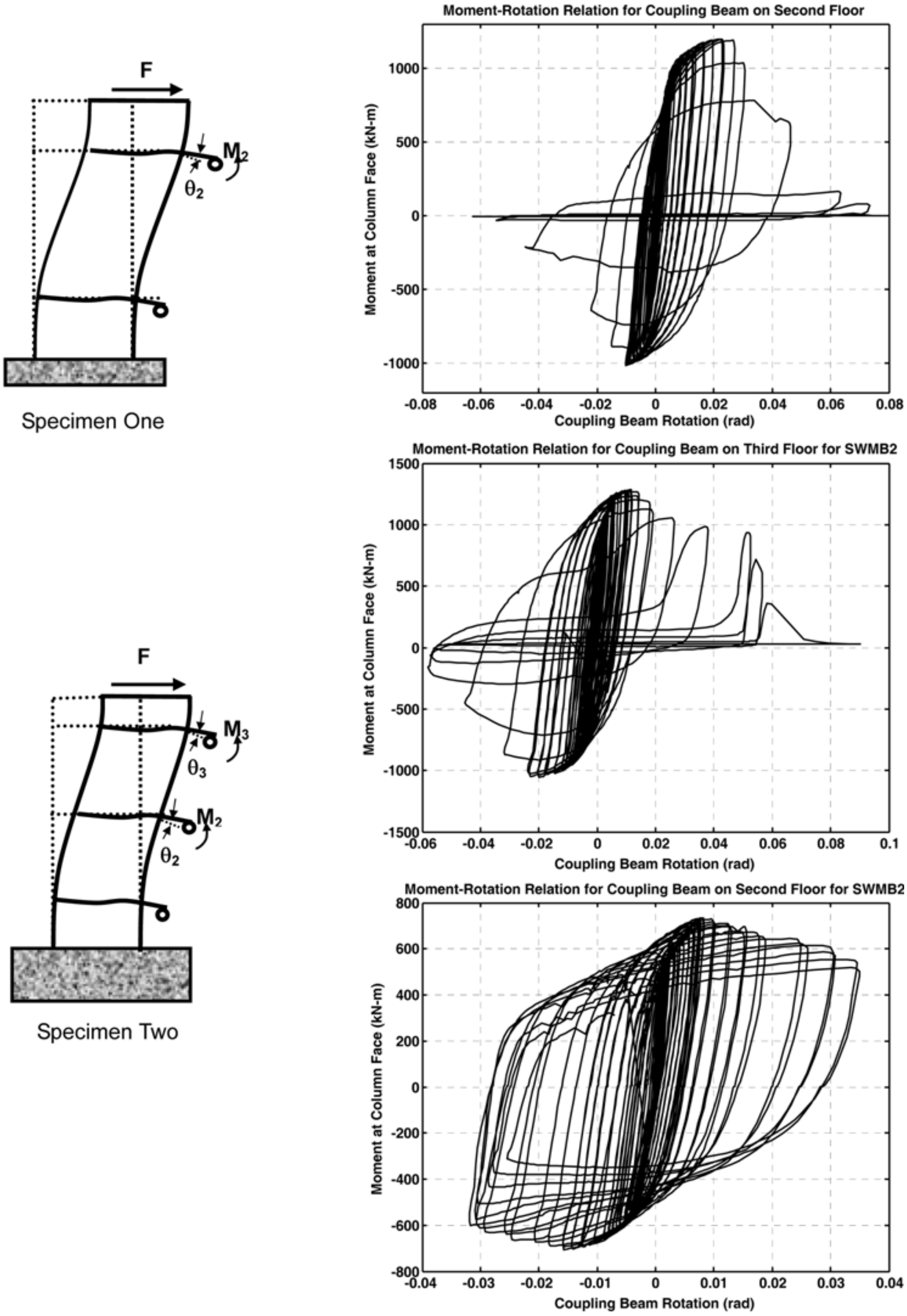

Figure 6. Hysteresis Behavior of Coupling Beams for Both Specimens.

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. Therefore an R-factor of 8.0 is suggested for the tested system.

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.

4. Application of Smart Structures Technology

With significant advances in materials, sensing and control technologies, research in application of smart structures technology to civil infrastructure design and

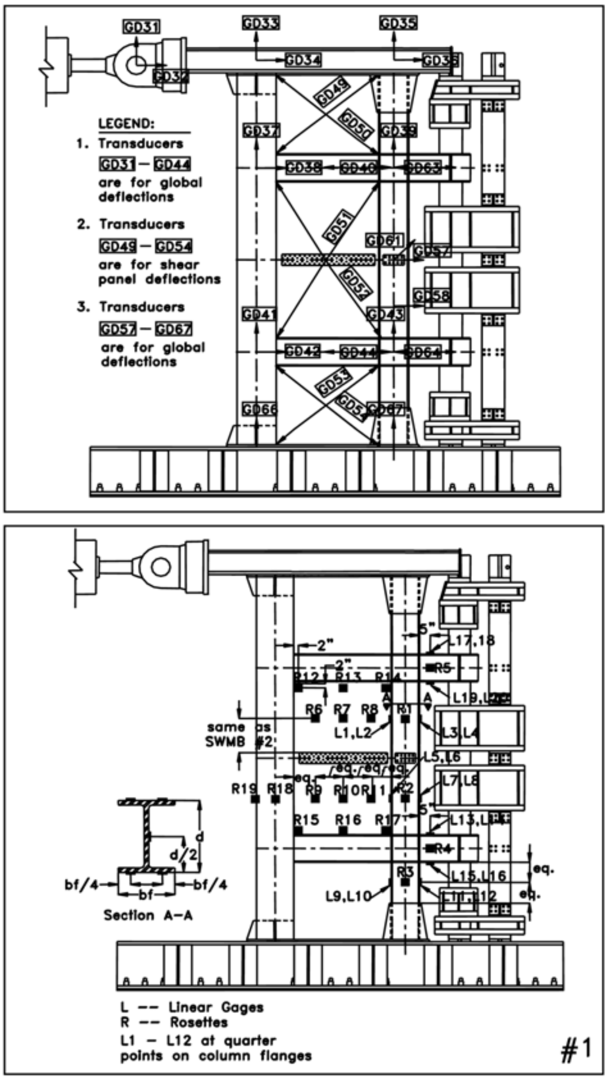

Figure 7. Instrumentation plan for steel shear wall specimen one.

construction has progressed a lot in the past decade, including structure theory, sensors, signal processing and control, actuators, etc. (Zhang, 2003; Maalej, 2002; Matsuzaki, 1997). This paper will discuss the potential application of smart structures technology into the design and construction of steel shear wall systems.

Structural health monitoring is one major sub-category under smart structures technology, and has been used in the design and construction of many long-span bridges. The sensors used on the bridges include accelerometers, strain gauges, displacement transducers, anemometers, thermometers, and level sensing stations. The sensors would provide essential information about the general health of a bridge and act as an early warning system (Zhang, 2003). Similar idea could be applied to steel shear wall systems, and monitoring of these systems would provide vital information on the general health condition of the high-rise buildings in which these systems are usually used as lateral load-resisting systems. Sensors used on these systems would include accelerometers, strain gauges, displacement transducers, etc. From the cyclic tests, it is clear that the most feasible locations for these sensors would be similar to the instrumentation plan. For example, the displacement transducers could be mounted at intersections of boundary columns and beams as well as corners of wall panels, and the strain gauges could be mounted at column bases as well as wall diagonal struts, as shown in Fig. 7. Data collected from these key locations would then be used in interpreting the behavior of the systems by the management group and help in their decision-making.

Other important applications of smart structures technology into the design and construction of steel shear wall systems could be detecting of the crack initiation at the steel wall corners and coupling beam-column connections since both test specimens failed due to low cycle fatigue, and introducing new materials for the CFT columns.

5. Conclusions

The projects described in this paper addressed the issues of cyclic behavior of one steel shear wall system, and proposed seismic design recommendations for the structural system. Through the experimental studies, it is clear that the behavior of steel shear wall system was very similar to behavior of steel plate girders subjected to shear, and the steel shear walls buckled along the compression diagonal and developed tension field along the tension diagonal after yielding. The system was very ductile under large cyclic displacements with maximum inter-story drifts over 3.2%. Therefore an R factor of 8.0 could be used in the seismic design. The experimental studies also showed the importance of keeping the gravity load carrying members in these systems intact under seismic effects, while the non-gravity carrying members could yield extensively and dissipate energy.

Smart structures technology could be applied to the design and construction of steel shear wall systems in monitoring their behaviors and provide early warning and data for decision-making.

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