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CYCLIC BEHAVIOR OF TRADITIONAL AND INNOVATIVE COMPOSITE SHEAR WALLS

Qiuhong ZHAO[1] and **Abolhassan ASTANEH-ASL**[2]

SUMMARY

Shear wall systems are one of the most commonly used lateral-load resisting systems in high-rise buildings. This paper concentrates on the experimental studies of two composite shear wall systems and presents a summary and discussion of test results.

Composite shear wall system studied herein consists of a steel boundary frame and a steel plate shear wall with a reinforced concrete wall attached to one side. The steel plate shear wall is welded to the boundary frame and connected to the reinforced concrete wall by bolts. In the system called "traditional" the reinforced concrete wall is in direct contact with the boundary steel frame, while in the system called "innovative" there is a gap in between.

Cyclic static tests were conducted on two half-scale specimens representing each system and both specimens showed highly ductile and stable inelastic behavior. The specimens were able to tolerate more than 17 cycles of inelastic shear displacements and reach maximum inter-story drift of more than 0.05. Inter-story drift herein is defined as lateral movement of the floor divided by the story height. Composite action of the steel and the reinforced concrete walls was ensured by the connecting bolts, which braced the steel plate shear wall and prevented its overall buckling. After shear yielding of steel plate, inelastic local buckling of steel plate shear wall occurred in the areas between the bolts. The experimental results and their implication in seismic design are summarized and discussed.

INTRODUCTION

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 many cases concrete shear walls are used with a boundary steel frame to resist seismic effects. However, there are several disadvantages for using a concrete shear wall in this case. The most important one is the development of tension cracks and localized compressive crushing during large cyclic displacement, which can result in spalling and splitting failure of the wall and lead to serious deterioration of stiffness and reduction in strength. Also, reinforced concrete shear walls used in tall buildings tend to develop relatively large shear forces during seismic

[1] Doctoral Graduate Student, Dept. of Civil and Environmental Engineering, 781 Davis Hall, University of California, Berkeley, CA 94720-1710, E-mail: qhzhao@berkeley.edu.
[2] Professor, Dept. of Civil and Environmental Engineering, 781 Davis Hall, University of California, Berkeley, CA 94720-1710, E-mail: astaneh@ce.berkeley.edu, Web: www.ce.berkeley.edu/~astaneh.

events due to their relatively large lateral stiffness, but the high weight to strength ratio of concrete material will make the use of reinforced concrete shear walls impractical for this case. In addition, the casting and curing of reinforced concrete walls in a steel building makes the construction not so efficient compared to other systems such as braced frames or moment frames.

In recent years, steel plate shear wall has been used in a number of buildings and achieved satisfactory results regarding construction efficiency and economy. However, overall buckling of the steel plate shear wall will result in reduction of shear strength, stiffness and energy dissipation capacity of the whole system (Zhao [1]). It could be prevented by adding stiffeners to the steel plate as often done in Japan, which, however, will result in additional fabrication costs (Astaneh [2]). In addition, in structures with steel shear walls, due to relatively large inelastic deformations of the panel, the connections of the boundary frame can undergo relatively large cyclic rotations as well as somewhat larger inter-story drifts (Allen [3]).

Composite shear walls, on the other hand, might combine the advantages of the reinforced concrete shear wall and steel shear wall together and promote the usage of shear wall systems in steel buildings. The composite shear walls have been used in a few modern buildings in recent years including a major hospital in San Francisco (Dean [4]), but not as frequently as the other lateral load resisting systems. Seismic behavior of this system and the design guidelines are therefore of high interest.

BACKGROUND

Traditional and Innovative Composite Shear Wall Systems

Four common types of composite shear wall systems are shown in Figure 1. The focus of this study was the behavior of type I system, which consists of a steel boundary frame and a steel plate shear wall with a reinforced concrete wall attached to one side using mechanical connectors such as shear studs or bolts. In the AISC Seismic Provisions this type of system is denoted as “Composite Steel Plate Shear Walls, (C-SPW)” (AISC [5]). Main components of this type of system are also shown in Figure 2.

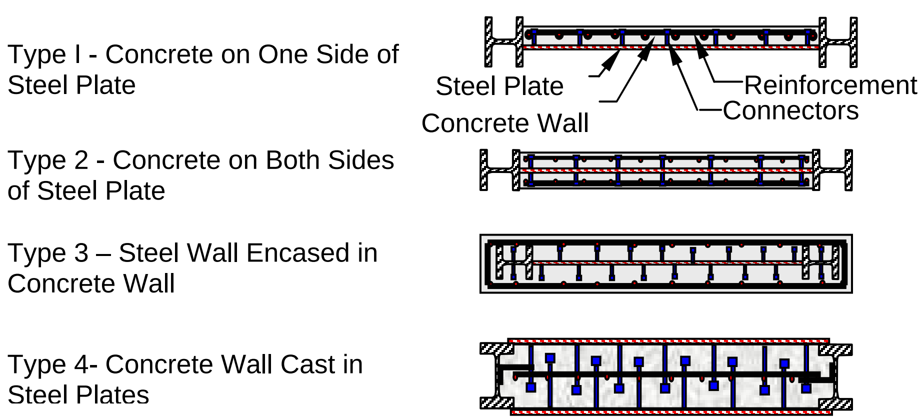


Figure 1. Composite Shear Wall Systems (Astaneh [6])

The study was focused on two configurations of type I composite shear wall system denoted as “traditional” and “innovative”. The innovative system was proposed and designed by the second author (Astaneh [6]). The difference between the two configurations is that in the traditional system the reinforced concrete wall is in direct contact with the steel boundary frame but in the innovative system there is a gap, as shown in Figure 2. The gap can be left empty or filled with soft material such as Styrofoam which was used in this project. If one desires to add to energy dissipation capacity of the structure at additional cost, the gap can be filled with visco-elastic material.

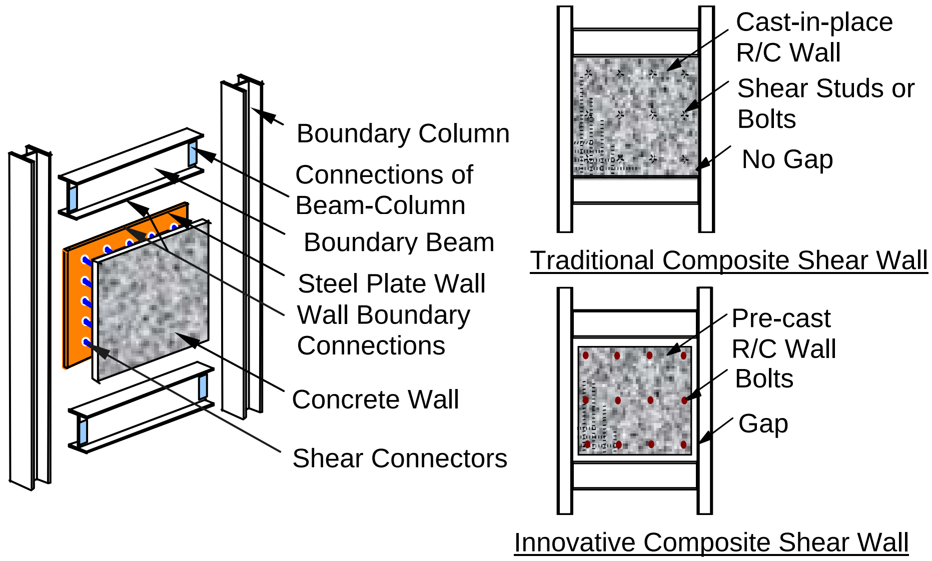


Figure 2. Main Components of Type I Composite Shear Wall (Zhao [7])

It is anticipated that the existence of the gap in the innovative composite shear wall system will result in reduction of the system lateral stiffness and change of the reinforced concrete wall behavior under severer seismic events. In the traditional system, both steel and concrete walls are active and provide stiffness and strength from beginning of loading. As a result, not only large forces can be attracted to the structure due to relatively large stiffness of the combined system, but the reinforced concrete wall could be damaged under relatively small lateral displacement. In the innovative system, however, due to the existence of the gap, the concrete wall will not get involved in resisting lateral loads until the inter-story drift has reached certain value, as shown in Figure 3.

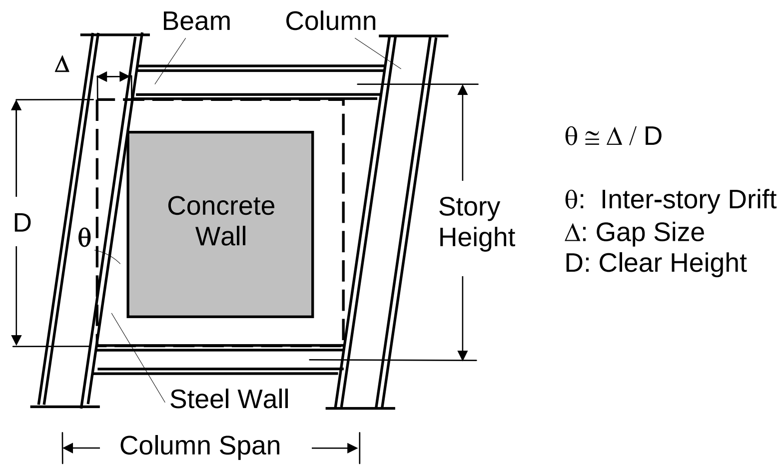


Figure 3. Function of Gap in the Innovative Composite Shear Wall System

When the drift is under the specified value, only steel shear wall and the boundary moment frame provide strength, stiffness and ductility, and the role of reinforced concrete wall is to provide out-of-plane bracing for the steel plate. When the drift is over the specified value, the gap is closed at corners and both steel and concrete walls become active and provide strength, stiffness and ductility. Then the participation of reinforced concrete wall brings in the much needed extra stiffness to help reduce the drift and P-Δ effects, compensates for loss of stiffness of steel shear wall due to yielding, and helps in preventing lateral creep and collapse failure of the structure due to P-Δ effects.

Objectives of the Study

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

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

CYCLIC TEST ON COMPOSITE SHEAR WALL SYSTEM

Specimens

The test program consisted of cyclic testing of two half-scale specimens. The specimens were constructed as sub-assemblies of a generic building over three floors with composite shear wall system as the lateral load resisting system, as shown in Figure 4. The test specimens represented a dual lateral load-resisting system where composite shear wall is the "Primary" lateral load resisting system welded to a "Back-up" system of special moment-resisting frame.

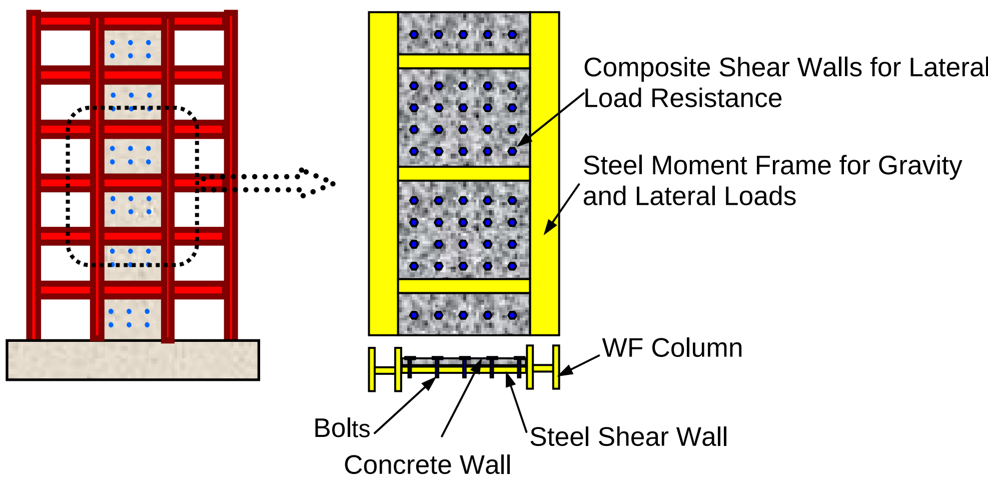


Figure 4. Generic Structures and Test Specimen

The two specimens had identical properties except that in Specimen One there was a 32 mm (1.25 inch) gap between the reinforced concrete wall panel and the surrounding steel frame in the two middle stories. Hence Specimen One was representing the "Innovative" Composite Shear Wall System and Specimen Two was representing the "Traditional" Composite Shear Wall System. The major components of test specimens are shown in Table 1 and Figure 5.

In the specimens, the steel wall plate was made of A36 with specified yield stress of 248 MPa (36 ksi). The steel boundary frame was made of A572 Grade 50 steel, with specified yield stress of 345 MPa (50 ksi). The reinforced concrete (R/C) shear wall was made of concrete with a specified f'c of 28 MPa (4000 psi) and one layer of reinforcement with a grid of #3 rebar and #5 rebar at perimeter. The R/C walls in the specimen were pre-cast and bolted to the steel walls by 13 mm (½ inch) diameter A325 bolts. The pre-cast reinforced concrete walls were cast in the lab.

Table 1. Components of Test Specimens

Steel Wall Plate Thickness	Pre-cast R/C wall				Wall Bolts Dia.	Beam Section*	Column Section*
	Thickenss	Rebar Dia.	Rebar Spacing	Reinf. Ratio			
4.8 mm (3/16 inch)	76 mm (3 inch)	10 mm (3/8 inch)	102 mm (4 inch)	0.92%	13 mm (½ inch)	W12x26	W12x120

* Properties of cross sections refer to the AISC Manuals (AISC [5]).

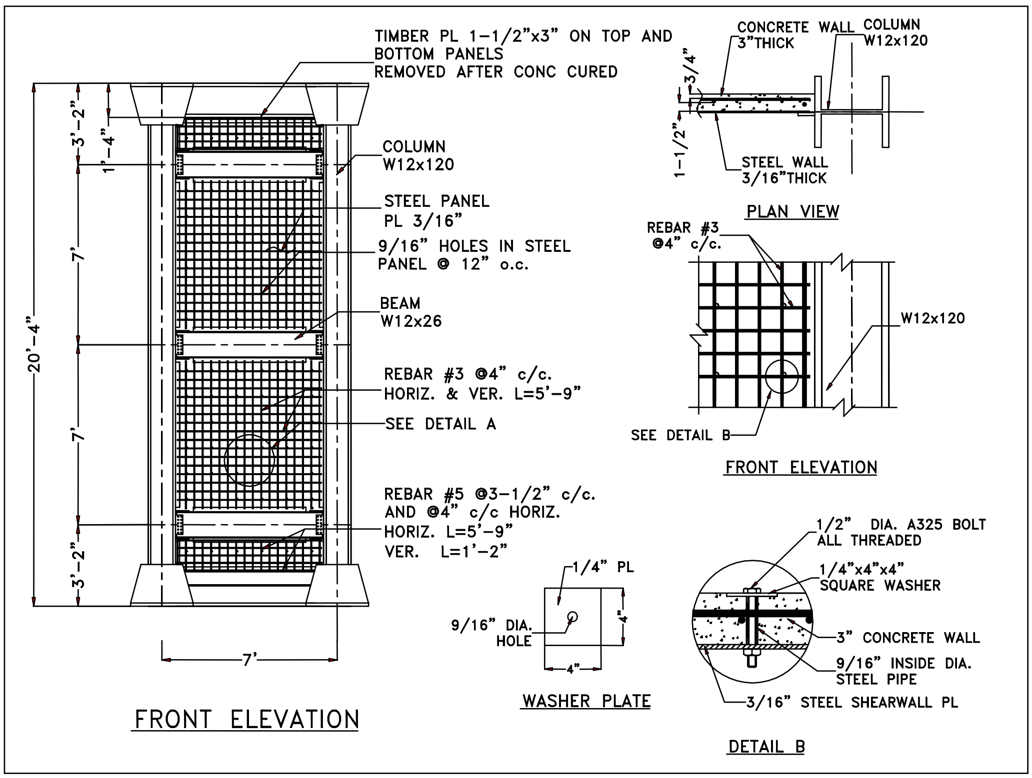


Figure 5. A Specimen with Details of Concrete Shear Wall

The steel parts were fabricated and welded by the fabricator. The steel wall plates were first bolted to the fish plates around the inner perimeter of the boundary frame to make sure they were in place, and then shop-welded to the surrounding steel beams and columns. Welds were all fillet welds using flux cored arc welds with E70T-7 electrodes and a specified strength of 483MPa (70 ksi).

The beam-column connection in the steel frame was cover plate plus shear tab moment connection designed according to FEMA 350 recommendations to make sure that the shear and plastic moment capacity of the beam will be fully developed (FEMA [8]). Some details were modified to accommodate the steel wall plate welded to the moment connection, and further increase the connection rotational ductility. For instance, a vertical fish plate was welded to each cover plate to hold the adjacent steel shear wall in place, and the free length of the cover plates was increased to make the connection even more ductile. Figure 6 shows the details of the moment connections.

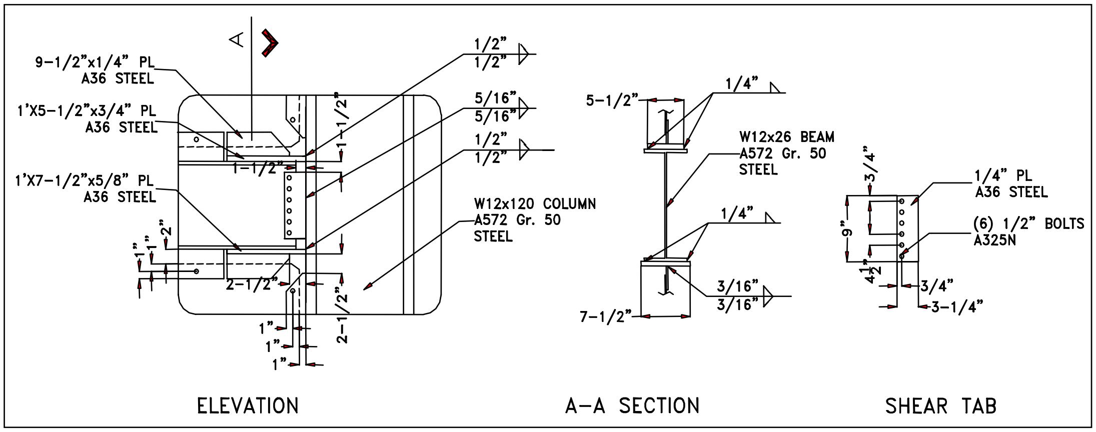


Figure 6. Beam to Column Moment Connections

Besides the major structural components, the top and bottom of the test specimens were strengthen to secure their connection to the test set-up and make sure premature local failure won't occur.

Test Set-up

The test set-up is shown in Figure 7. Main components of the test set-up are: Actuator, Top Loading Beam, Bottom Reaction Beam, R/C Reaction Blocks, and Bracings. The test set-up was designed so that lateral displacements and forces could be applied to the specimen and sufficient factor of safety could be provided under the large forces generated. The test set-up also provided the boundary conditions for the specimen that resembled the actual boundary conditions for a typical floor in a generic structure.

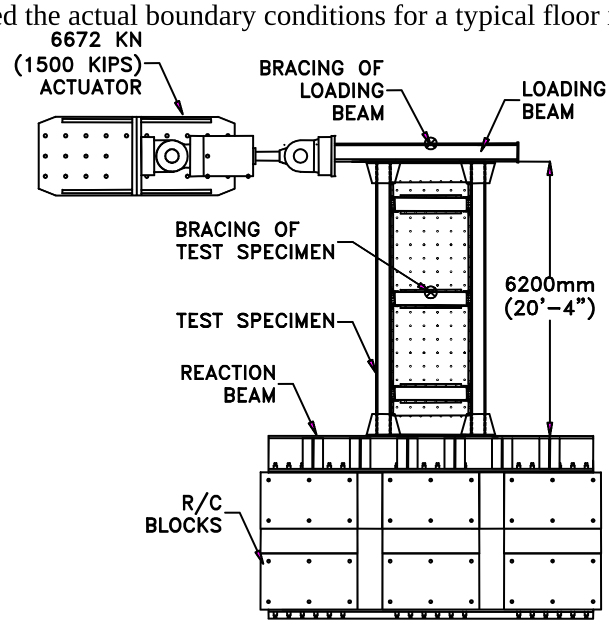


Figure 7. Test Specimen and Test Set-up

The actuator could provide ±305 mm (±12 inch) of maximum displacement and ± 6672 kN (±1,500 kips) of maximum push-pull force. It was fixed to the laboratory floor using 51 mm (2inch) diameter pre-stressing rods. A cyclic displacement was applied by the actuator to the top of the specimen along with the cyclic load, which resulted in the same story shear for all the stories in the specimen. It represented the shear distribution in the generic structure, since composite shear wall systems are more economical to use in tall buildings where variation of shear forces between adjacent stories are usually very small (Zhao [1]). Gravity loading was ignored since gravity didn't play an important role in the seismic behavior of composite shear walls.

The Loading Beam was connected to the actuator loading-arm at one end and attached to the top end plate of the specimen longitudinally. The Reaction Beam was longitudinally attached to the bottom end plate of the specimen and the R/C Reaction Blocks. The Reaction Blocks were seven reinforced concrete blocks pre-stressed together and to the laboratory floor. The actuator lateral force was then transferred following the path: Actuator – Loading Beam – Specimen – Reaction Beam – R/C Reaction Blocks – Test Floor. In order to simulate bracing effects provided by the floors and prevent out-of-plane movements, bracings were applied at two locations – one to the top flange of Loading Beam, and the other to the top flange of the middle beam of specimen.

Loading History

Both tests used the same loading history established according to the specifications for Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections in Seismic Provisions for Structural Steel Buildings (AISC [5]). The loading sequence and major events for both specimens are shown in Figure 8. Notice here the loading sequence is given in terms of overall drift, which is defined as the lateral displacement of the actuator divided by the total height of specimen.

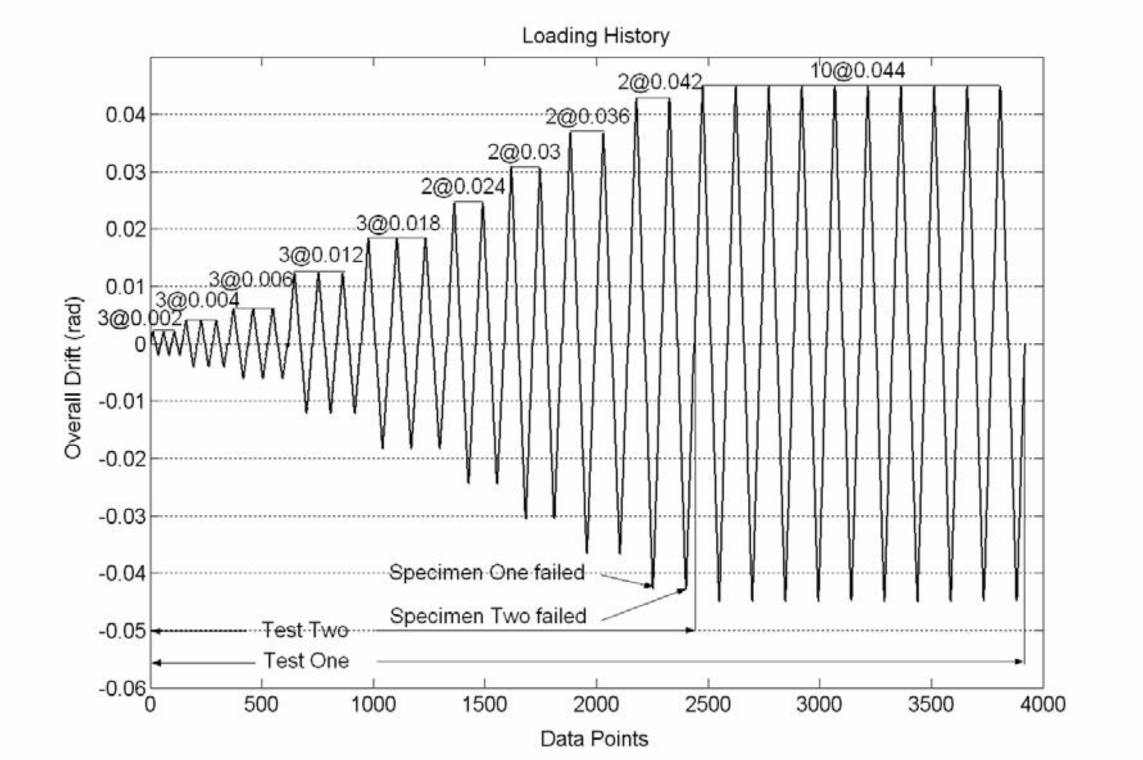


Figure 8. Loading History Applied to Both Specimens

Instrumentation

A number of Linear Variable Displacement Transducers (LVDT's) were put on the specimens to measure the global as well as local displacements 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, rotation of a beam in a moment connections could be measured. Linear as well as "rosette" strain gages were also mounted to the specimen at various critical locations including flange and web of the column, middle point of the steel panel, quarter point of the steel panel etc. For safety purpose, a series of displacement transducers were mounted to the test set-up to detect any slippage of the actuator or the reaction blocks relative to the test floor. The loading system will be automatically shut down if the slippage exceeds 3mm (1/8 inch).

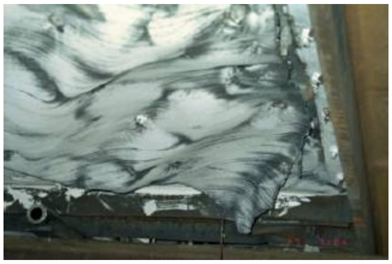
CYCLIC BEHAVIOR OF COMPOSITE SHEAR WALL SPECIMENS

Innovative Composite Shear Wall Specimen

Specimen One, with a 32 mm (1.25-in) gap around the R/C panel, behaved in a very ductile and desirable manner. Up to overall drifts of about 0.006, the specimen was almost elastic. At this drift level some yield lines appeared on the beams as well as column base. At overall drifts of about 0.012, the compression diagonal in the steel wall panels was buckling and diagonal tension field was forming. The specimen could tolerate 33 cycles, out of which 27 cycles were inelastic, before reaching an overall drift of 0.044 and maximum shear strength of about 2790 kN (627 kips). At this level of drift, fractures were widespread in the walls and frame members due to low-cycle fatigue, and the connectors between steel wall and concrete wall were almost gone. Shear strength of the specimen dropped to about 80% of the maximum capacity of the specimen, and the specimen was considered failed. Key test observation of Specimen One is summarized in Table 2. Figure 9 shows appearance of different components of Specimen One at the end of the test.

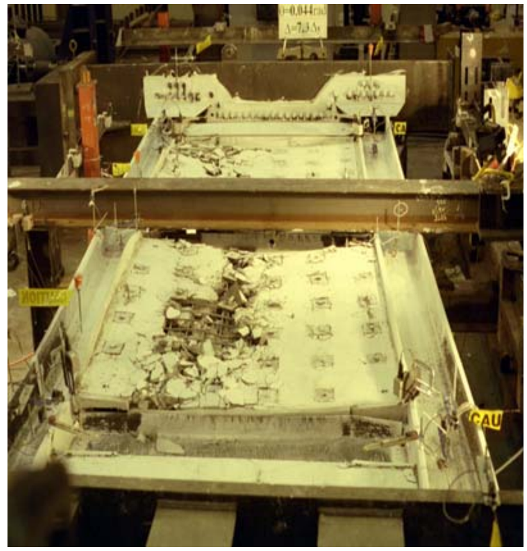
Table 2. Key Test Observations on Behavior of Specimen One

Loading Groups	Actuator Drift (Rad)	Description
0	0.0002	Very small warm-up cycle.
1	0.002	Specimen remained elastic.
2	0.004	Specimen remained elastic. Slight yielding at base plate. **"Proportional Limit Point".**
3	0.006	All three beams and column base yielded. **"Significant Yield Point".**
4	0.012	Steel shear wall locally buckled in compression and yielded in tension diagonally. Minor damage to RC wall. Shear tabs in beams yielded.
5	0.018	Column flange and web yielded at base. Lower steel wall formed buckling shape of "X".
6	0.024	Middle beam web and bottom beam flange locally buckled. Top steel wall diagonally buckled and formed kinks at quarter points. Diagonal cracks in lower RC wall.
7	0.03	Some bolts between walls punched through the steel plate. Cracks formed at the corners of steel wall. **"Point of Maximum Shear Strength".**
8	0.036	Both RC walls developed major cracks and crushed at the corners. Middle beam web fractured at the south end.
9	0.042	Specimen failed by fracture of steel wall plate. 10% of the bolts between walls failed. RC walls crushed heavily and lifted around edges. Fracture of beam flange. **"Point of Maximum Ductility".**
10	0.044	Heavy column flange local buckling. Fractures on all beam webs. Severe fracture of steel wall panel at corners. RC walls spalled, and rebar buckled.



Fracture of Steel wall

Yielding of Column Base



Crushing of Concrete Wall

Figure 9. Specimen One at 7.3 δy (Overall Drift of 0.044)

Traditional Composite Shear Wall Specimen

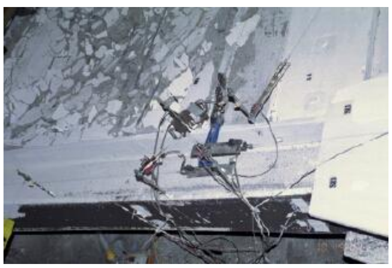
Specimen Two also behaved in a ductile manner. Up to inter-story drifts of about 0.006, the specimen was almost elastic. At this drift level some yield lines appeared on the bottom and middle beam webs as well as column base plate. The specimen was able to reach cyclic overall drift of 0.042 after undergoing 23 cycles, 17 of the cycles being inelastic. The maximum shear force reached was about 3020 kN (679 kips) during the 19th cycle. Throughout the test, the gravity load carrying column remained essentially stable while non-gravity carrying lateral load resisting elements underwent well-distributed and desirable yielding. During 23rd cycle, the upper steel shear wall plate fractured totally along the north and bottom edges due to low-cycle fatigue, and the connectors between steel wall and concrete wall were almost gone. Shear strength of the specimen dropped to about 80% of the maximum capacity of the specimen, and the specimen was considered failed. Key test observation on Specimen Two is summarized in Table 3. Figure 10 shows appearance of different components of Specimen Two at the end of the test.

Table 3. Key Test Observations on Behavior of Specimen Two

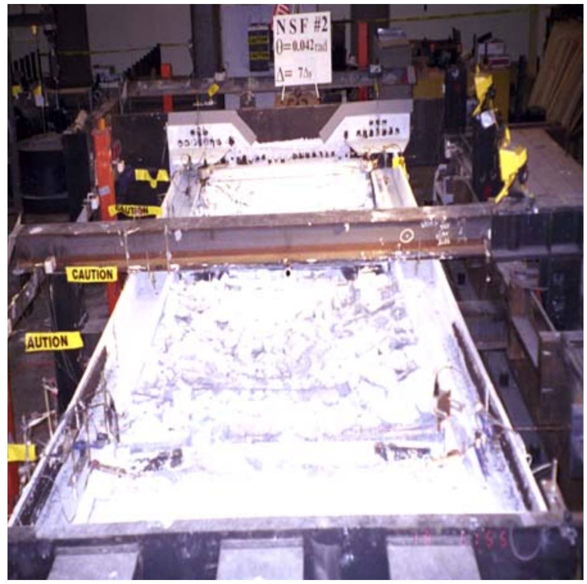
Loading Groups	**Actuator Drift (Rad)**	**Description**
0	0.0002	Very small warm-up cycle.
1	0.002	Specimen remained elastic.
2	0.004	Specimen remained elastic. Slight yielding at bottom beam web. **"Proportional Limit Point".**
3	0.006	Bottom and middle beam web and column base plates yielded. **"Significant Yield Point".**
4	0.012	Yielding in steel shear wall, but no obvious buckling. RC wall separated from steel frame. Much yielding of beam webs and shear tabs. Heavy yielding of column flanges at base.
5	0.018	Cracks in RC wall. Steel wall diagonally buckled. Yielding in top beam web. One bolt connecting the walls in second story sheared off. **"Point of Maximum Shear Strength".**
6	0.024	RC walls crushed around perimeters. Lower panel lifted about 3 inch. Noticeable web local buckling in middle beam.
7	0.03	Cracks in steel wall near corners in lower panel. First punching failure of bolts

		connecting the walls. Flange of middle beam locally buckled.
8	0.036	RC walls had major cracks and crushed at corners. Severe web and flange local buckling on all beams, and web fracture in middle beam.
9	0.042	Specimen failed by upper steel shear wall torn off from north column along entire height. RC crushed to rubble. More than 30% of total bolts failed. Middle beam flange and web fractured. **"Point of Maximum Ductility".**

Fracture of Moment Connection



Yielding of Column Base



Crushing of Concrete Wall

Figure 10. Specimen Two at 7δy (Overall Drift of 0.042)

DISCUSSION ON TEST RESULTS

Basically, both specimens showed high ductility during the tests since both of them reached a maximum inter-story drift of at least 0.05 before their shear strength dropped to 80% of the maximum capacity. Inter-story drift herein is defined as lateral movement of the floor divided by the story height. In both cases, failure of the specimen was caused by fracture of the steel shear wall. The columns in the steel moment frame behaved in a quite stable manner until failure with only some flange local buckling at an overall drift of more than 0.04, which means the gravity system still had enough strength and stiffness under seismic effects and progressive collapse was not likely to occur.

Maximum shear response of both specimens under cyclic displacements was given in Table 4, from which we could see that both specimens had the same yield point of 0.006 overall drift, as predicted by the pre-test inelastic analysis. As observed during the tests, in both cases the yielding of the whole system was caused by yielding of column flange at the base and beam webs. Therefore the behaviors of both specimens before yielding were quite similar.

It was also clear from the table that under the same lateral displacement, the maximum shear response and secant stiffness of Specimen One were always slightly lower than Specimen Two, until it reached the maximum shear strength at an overall drift of 0.03. The behavior was reasonable considering the participation of the reinforced concrete wall of Specimen Two in resisting shear from the very first beginning. While in Specimen One the participation was delayed due to the gap between the wall and the steel frame. However, the difference was never beyond 25%, which indicated that the reinforced concrete wall did not contribute very much to the strength and stiffness of the specimen. Instead, it mainly provided bracing for the steel wall and prevented the steel plate from overall buckling before yielding, as observed in the early cycles of both tests.

Table 4. Maximum Base Shear Response of Test Specimens

Loading Groups	Actuator Drift (Rad)	Max. Base Shear Response V				
		V^I (Specimen One)	% V^I_{max}	V^{II} (Specimen Two)	% V^{II}_{max}	V^{II} / V^I
1	0.002	712 kN (160 kips)	26%	729 kN (164 kips)	24%	103%
2	0.004	1290 kN (290 kips)	46%	1414 kN (318 kips)	47%	110%
3	0.006	1717 kN (386 kips)(1)	62%	1953 kN (439 kips) (1)	65%	114%
4	0.012	2264 kN (509 kips)	81%	2789 kN (627 kips)	92%	123%
5	0.018	2589 kN (582 kips)	93%	3020 kN (679 kips) (2)	100%	117%
6	0.024	2767 kN (622 kips)	99%	2971 kN (668 kips)	98%	107%
7	0.03	2789 kN (627 kips)(2)	100%	2976 kN (669 kips)	98%	107%
8	0.036	2758 kN (620 kips)	99%	2673 kN (601 kips)	89%	97%
9	0.042	2526 kN (568 kips)	91%	2397 kN (539 kips) (3)	79%	95%
10	0.044	2197 kN (494 kips)(3)	79%	N.A		N.A

Note: (1) Specimen Yield Shear Strength V^{II} / V^{I}= 114%

(2) Specimen Ultimate Shear Strength V^{II} / V^{I}= 108%

(3) Specimen Failure Shear Strength V^{II} / V^{I}= 109%

However, in the later stages of the test and after Specimen One reached its maximum shear strength, the maximum base shear response of Specimen One was slightly higher than (within 5%) Specimen Two under the same lateral displacement. According to test observations, in those loading cycles, damage to the concrete wall panels in Specimen Two became much severe, and concrete almost crushed to rubbles at the end of the test. While in Specimen One the damage to the concrete panels was moderate and concrete panels were helping in resisting shear since the gap between the panels and the boundary frame were already closed under such large displacements. Therefore the maximum base shear in Specimen One was slightly higher than Specimen Two under very large displacements.

Figure 11 shows shear force- drift curves for the third floor of both specimens. The energy dissipation curves for the same story were also shown. From the plots we can see that there is not much difference in the total shear strength or the energy dissipated by the third story for both tests. However, since the damage to the concrete walls in Specimen Two was much severe, the shear force- drift envelope curve for Specimen Two was much less smooth and stable than Specimen One. Therefore there is not much benefit if concrete wall was engaged in resisting the lateral-force from very early stages.

The R/C walls behaved in a much different manner in the two tests. Under the same overall drift of 0.018, the perimeter of the R/C wall in Specimen Two (the one without the gap) started to crack and spall, while the R/C wall in Specimen One (the one with a gap) did not show much damage. The damage to the R/C wall and connecting bolts were always less severe for Specimen One when subjected to the same lateral displacement. At the point of system failure, the damage to the R/C wall in Specimen Two was so extensive that most of the concrete had crushed and turned into rubble with reinforcement grid almost entirely unattached and exposed, while in Specimen One both panels could keep most of their shapes.

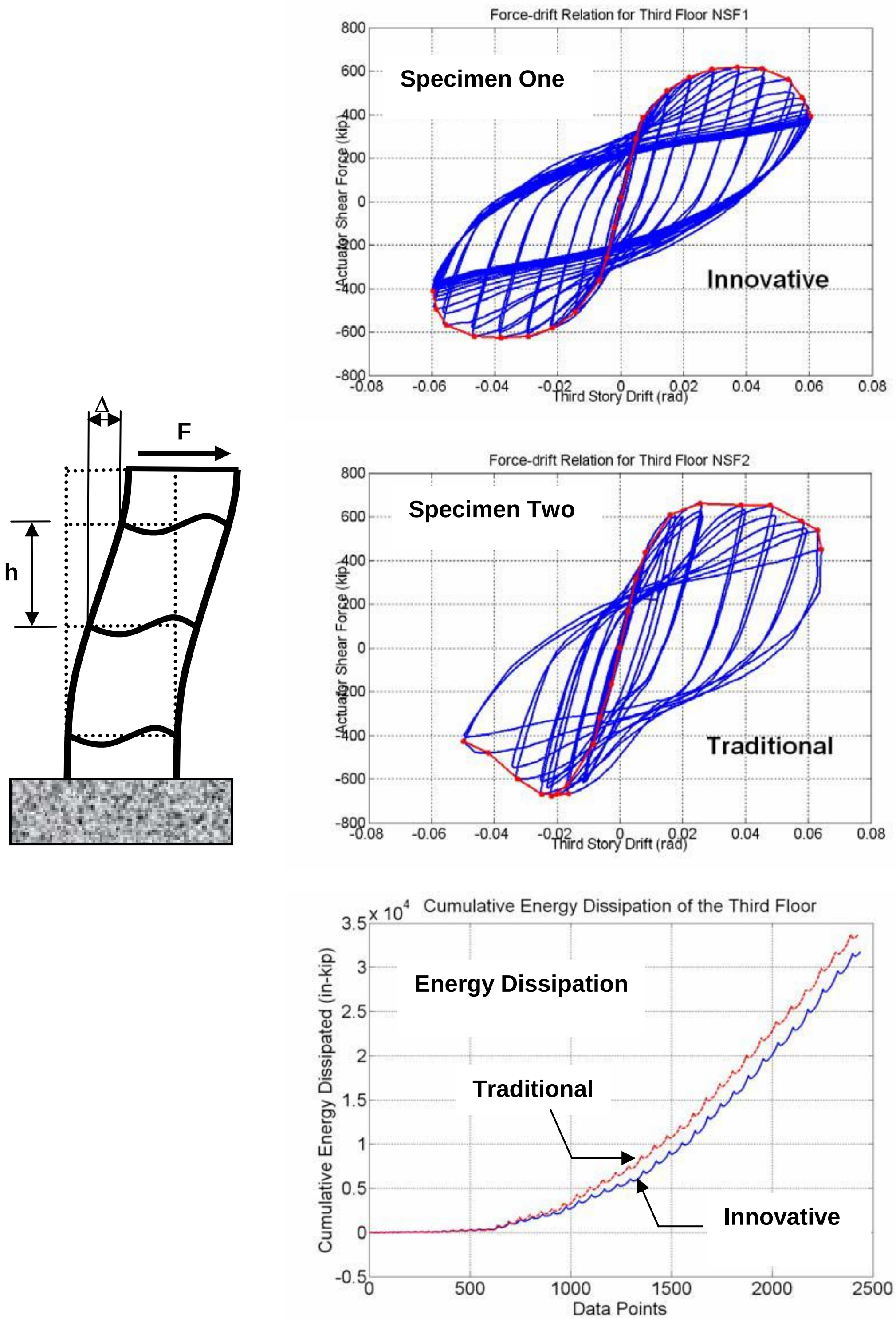


Figure 11. Shear Force- Displacement and Energy Dissipation Curves for Third Floor

In the two tests, the steel shear walls behaved in a similar manner. The steel walls were intact in the early cycles up to the yield point of 0.006 overall drift. During overall drift of 0.012 to 0.018, the steel plates locally buckled over the free length between the connecting bolts. After that, the wall plates continued to buckle and the connecting bolts started to punch through the wall panel. In both specimens, the steel wall plate started to develop cracks from the corner at about 0.03 overall drift, which finally developed into major cracks and caused the failure of the system at the end. The cracks were most likely caused by the discontinuity of the wall plate at the corner in the shape of a 25mm by 64mm (1 inch by 2.5inch) gap between the steel wall and the moment connections. Decreasing such discontinuity in the connection area would therefore be a major improvement in the design and construction of such shear wall systems. Plots of diagonal strains on the third floor of both specimens were given in Figure 12.

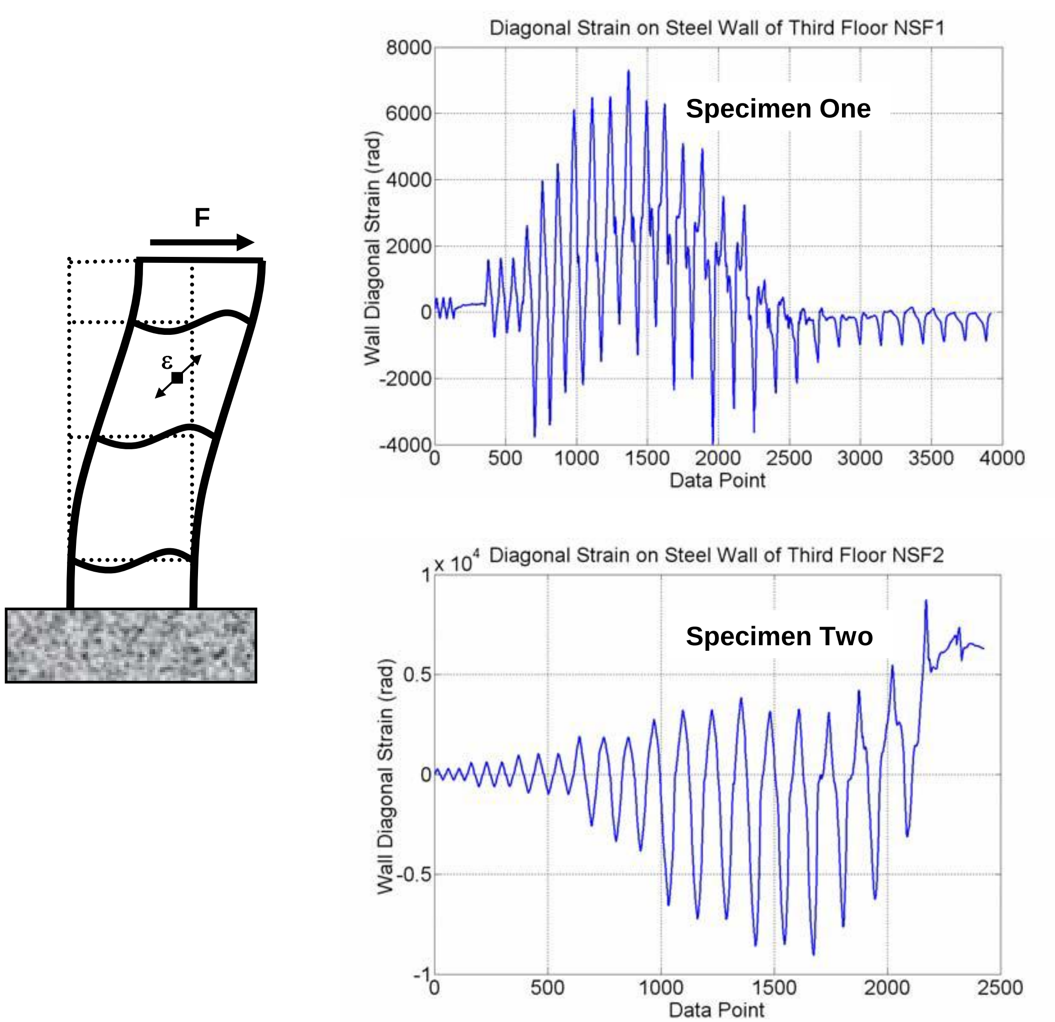


Figure 12. Diagonal Strain on Third Floor of Two Specimens

In summary, both Traditional and Innovative Composite Shear Walls were structural systems with high ductility and good lateral-load resisting properties. The maximum inter-story drift could reach more than 0.05 for both specimens and both could tolerate an inter-story drift up to 0.04 without reduction in the shear strength. The two specimens behaved in a similar manner in the elastic range with the same yield point. The R/C wall in both specimens did not contribute much in increasing the shear strength, but it was essential in bracing the steel plate wall and preventing it from buckling before the whole structure yielded. The R/C wall received much less damage in Specimen One than Specimen Two and so did the connecting bolts. The steel walls failed in fracture starting from the corners due to geometric discontinuity of the plates in that area. The fracture could be further delayed by eliminating this discontinuity.

CONCLUSIONS AND APPLICATION IN SEISMIC DESIGN

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

1) Both specimens performed in a very ductile manner and were able to reach inter-story drift ratio of 0.05 before the shear strength dropped below 80% of the maximum shear force reached during testing. The tests indicated that the system is an efficient lateral load resisting system with significant ductility and energy dissipation capacity.

2) Both specimens had the same yield point. Specimen that represents the Innovative Composite Shear Wall System behaved in more ductile manner than the specimen representing the traditional system. The shear strength and stiffness of the traditional system were slightly higher.

3) By introducing the gap in the innovative system, damage to concrete wall under relatively large cycles was much less than the damage to concrete wall in traditional system.

4) The gravity carrying elements- the WF columns in the steel boundary frame behaved stably until failure.

5) The lateral load resisting elements – steel wall plates and the WF beams yielded extensively. Specimen failed after the fracture of steel shear wall.

6) The proposed innovative composite shear wall underwent five phases of behavior:
   (a) Elastic behavior where only steel shear wall and boundary moment frame were active.

   (b) First inelastic phase where steel shear wall and limited areas of beams and columns were yielding. During this phase, due to presence of gap around concrete wall, the R/C wall was primarily acting as bracing element for steel plate preventing it from overall buckling.

   (c) When due to relatively large drifts, the gap between the concrete wall and boundary frame was closed, the steel plate and concrete wall acting as composite shear wall were both participating in providing strength and stiffness to the system.

   (d) During the final stage, after concrete wall was heavily damaged, the steel wall was acting as unstiffened shear wall and was buckling along compression diagonal and primarily resisting shear through tension field action.

   (e) During the final stage and after steel shear wall had also been damaged, the boundary steel moment frame was acting as a ductile moment frame.

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