CYCLIC BEHAVIOR OF C-SHAPED COMPOSITE PLATE SHEAR WALLS – CONCRETE FILLED

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Abstract

A Composite Plate Shear Wall/Concrete Filled (C-PSW/CF) is a special lateral-force resisting system consisting a sandwiched panel of two steel plates with concrete infill in between them, ideally suited for core-wall structures in high-rise construction. The steel plates are connected to each other using tie bars that are embedded in the concrete infill and, in some instances, steel-headed stud anchors. This research project was conducted to investigate the cyclic lateral load behavior of these walls, in terms of strength, and drift capacity. The testing program includes two C-shaped and two T-shaped large-scale concrete filled composite plate shear core walls subjected to flexure and axial loads together. Focus of the current paper is on typical results for C-shaped walls. Dimensions of the C-shaped walls were the same, but different axial loads were applied up to 19% of axial loading capacity. The composite behavior and the plastic hinge development were investigated and compared to results from plastic moment calculations. This provides valuable results on the expected behavior of one composite cross-section that is frequently used in full core wall. This is done to support the development of design guidelines for high-rise core-wall steel buildings having C-PSW/CF as the primary lateral force resisting system.

Keywords: Composite plate shear walls; Cyclic testing; Inelastic Flexural behavior, Ductility, Strength degradation.

1. Introduction and Background

A Composite Plate Shear Wall/Concrete Filled (C-PSW/CF) is a special lateral-force resisting system consisting of a sandwiched panel of two steel plates with concrete infill in between them. The steel plates are connected to each other using tie bars that are embedded in the concrete infill and, in some instances, steel-headed stud anchors. C-PSW/CF is lateral load resisting systems ideally suited for core-wall structures in high-rise construction (as well as other shear wall applications). The structural system is particularly appealing for tall building construction [1]. While this structural system has been used in selected applications in the past [2, 3, 4] knowledge on its cyclic inelastic non-linear behavior is needed for application in regions where severe earthquakes are expected.

Some research has investigated in-plane cyclic inelastic behaviour [5, 6, 7, 8]. However, no research has been conducted on C-Shaped walls, especially when subjected to simultaneously applied axial and lateral loadings. This knowledge is necessary for the subsequent further development of design guidelines when high-rise core-wall steel buildings having C-PSW/CF are used as the primary lateral force resisting system.

With the above in mind, C-PSW/CFs were tested at the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at State University of New York (SUNY) at Buffalo. Early findings from this experimental program are presented here.

2. Test Specimens and Setup

A series of two cyclic large-scale tests were conducted on cantilever composite walls having C-Shaped cross-section in plan. The dimensions and properties of selected specimens are: wall height, H= 166in.; flange length, h= 97.5in.; web length, b= 30in.; steel plate thickness, t=0.1875in.; flange thickness, d= 6in.; web thickness, c= 8.375in.; tie spacing = 6in.; tie diameter= 0.5in.; wall aspect ratio, H/b = 5.53; cross-section aspect ratio, γ=b/h = 0.31; flange aspect ratio, α=d/h=0.06; web aspect ratio, β=c/b =0.28; steel area, A_s= 61.8in2; concrete area, A_c = 9225.2in2; reinforcement ratio of web, ρ_{web}= 4.5%; reinforcement ratio of flange, ρ_{flange}= 6.3%, and; reinforcement ratio, ρ_s = 6.3%. Each specimen was cast into a footing designed to transfer the maximum moment developed in the wall to the strong floor of the laboratory. For more information on the specimen strength and detailing, refer to [9].

The loading set-up (shown in Fig. 1) consisted of two vertically inclined actuators (oriented at about 70 degrees from the horizontal) used to apply an axial load to the specimen, and two lateral actuators to apply cyclic lateral loading. Also shown in Fig. 1 is a T-shaped wall representative of those subsequently tested, but for which experimental results were not available at the time of this writing.

Fig. 1 – Test setup of C-shaped (left) and T-shaped (right) composite walls

3. Loading Protocol

The cyclic testing protocol was designed based on the yield displacements obtained from the Finite Element Analysis (FEA) pushover of Specimen C1. Cycles were based on first yield displacements (Δ_y) in the early stages of testing, and on a bi-linearized push-over curve to define equivalent yield displacements (Δ_y') in subsequent cycles. Drifts were limited to 6% for safety reasons, to keep the specimen stable upon substantial strength degradation.

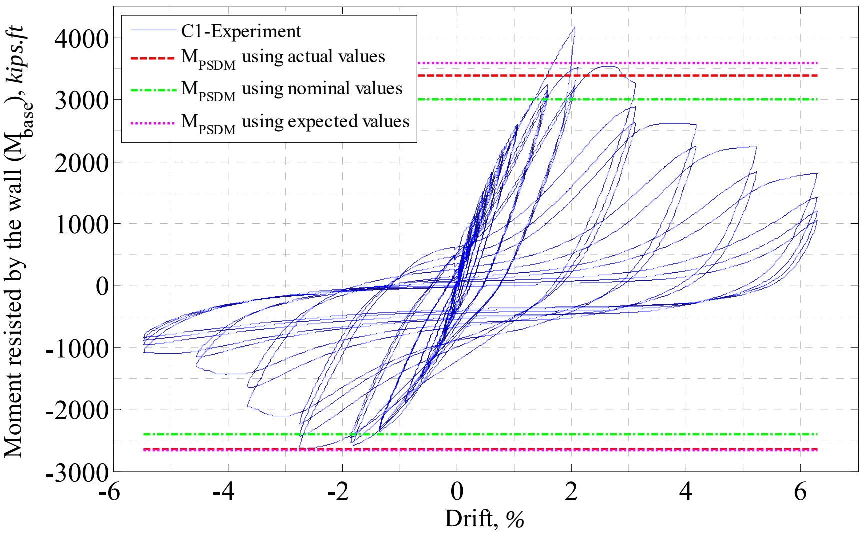
4. Application of Axial Loading on the Specimens

Axial loads equal to 15% and 19% of $A_c f'_c$ of each specimen were applied on Specimens C1 and C2 respectively. The centroid of the specimens is located at $\bar{y}$= 9.11in. from the outside face of the flange. However, for practical reasons in the test set-up design, the axial loading resultant was applied centered on the top of flange rather than at the section centroid. This distance between the centroid and center of the axial load resulted in a moment due to the eccentricity of the axial load, which was taken into account when post-processing the experimental results.

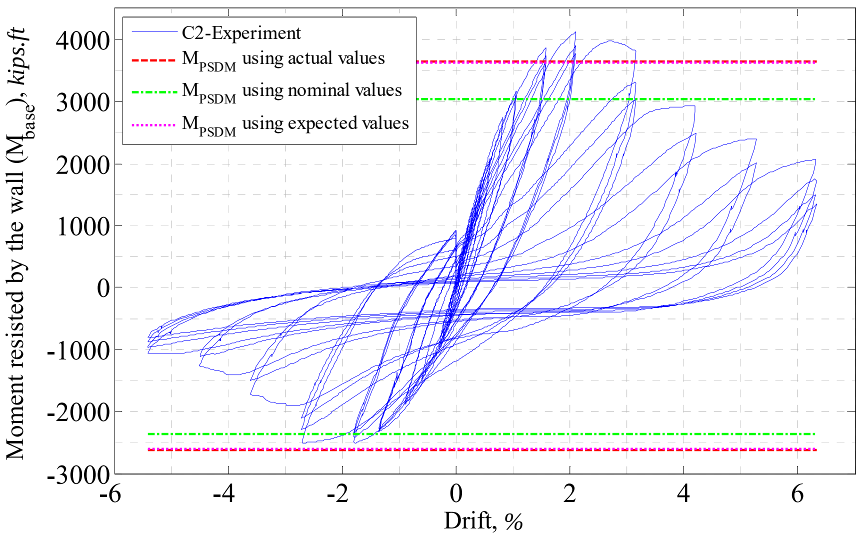
5. Test Results and Observations

The experimentally-obtained applied lateral force versus top lateral drifts for Specimens C1 and C2 are shown in Figs. 2a and 2b, respectively. Note that the moment resisted by the wall at its base was corrected to account for the eccentricity of the vertical load about the centroid of the wall cross-section, and for the resultant horizontal force from the vertically inclined actuators that the horizontal actuators resisted as the wall drifted during test. The peak recorded value for Specimen C1 is an outlier believed to be due to a recording error in the data acquisition system, and this value was not taken into account in the subsequent calculations when referring to wall strength. Such a spurious peak was not observed in the test result of Specimen C2.

Note that in these test, the drop in flexural strength is only attributable to the progressive development of fracture into the steel plates. Accelerated videos of the complete cyclic tests are to be presented during the 17WCEE conference, and will allow to better appreciate the development of local buckling and the fracture initiation and progressive propagation along the cross-section for these specimens.



(a)



(b)

Fig. 2 – Comparison of calculated theoretical resistance moments and the experimental base moment for: (a) Specimen C1, and; (b) Specimen C2

Taking the effective yield displacement as the displacement corresponding to the intersection of a line tangent to the initial slope of the resulting pushover results and a horizontal line set at the level of the maximum base moment obtained from test, ductility was calculated to be 4.02 and -4.3 in the positive and negative directions when flexural strength dropped to 80% of the peak value developed.

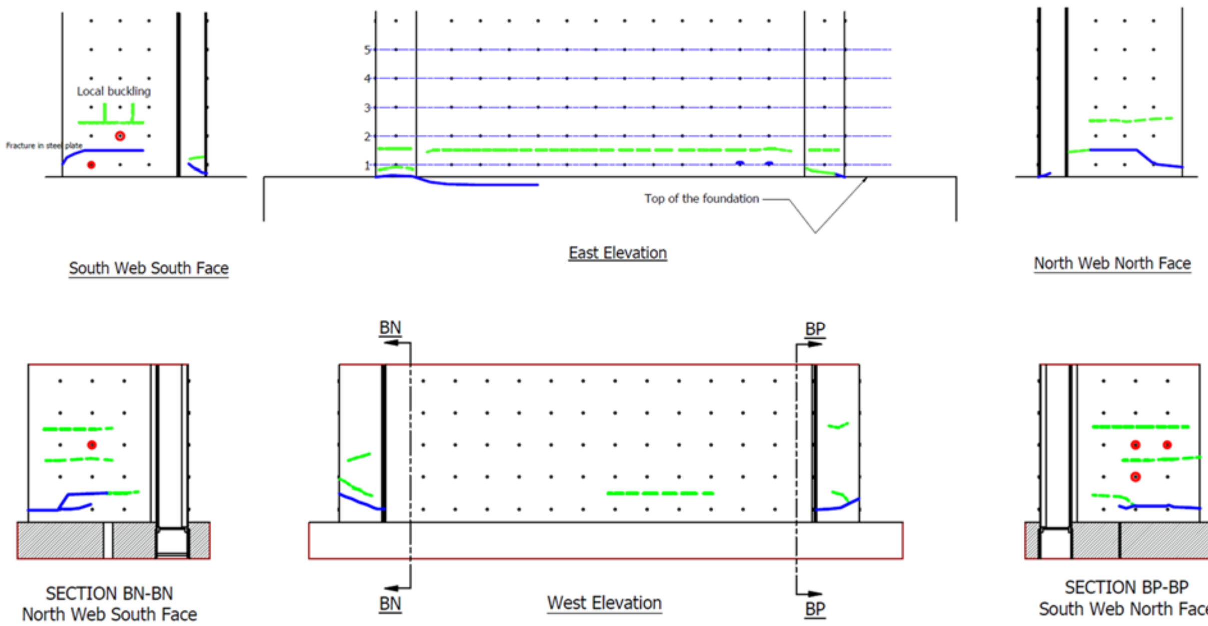
Table 1 summarizes the actual, nominal, and expected material properties and calculated flexural resistances (accounting for the applied axial load) for Specimens C1 and C2. The actual strengths are calculated using values obtained from the testing of steel coupons of samples from the wall's web and flanges, and of concrete cylinders cast during construction of the walls and tested on the corresponding specimen test day. The nominal strengths are calculated using the specified yield values for the steel plates, namely 50ksi for the A572Gr50 steel, and for the concrete ordered as a 4ksi concrete from the supplier. The expected strengths are calculated using the specified steel yield and concrete compressive strengths multiplied by R_y=1.1 and R_c=1.5×0.85, respectively.

The experimentally obtained moments developing at the base of the walls were also compared to their theoretical strength calculated using the Plastic Stress Distribution Method (PSDM). The theoretical values were calculated using the actual, nominal, and expected material properties, as described above. Comparisons are shown in Figs. 2a and 2b above for Specimens C1 and C2, respectively.

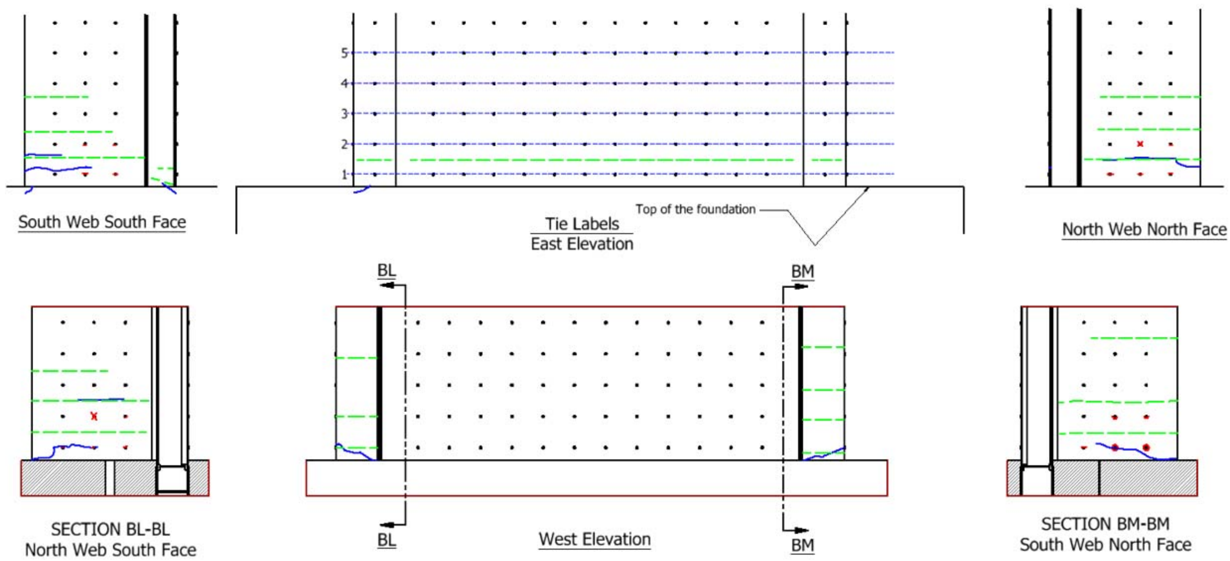
Table 1. Actual, nominal, and expected material properties and calculated flexural resistances for Specimens C1 and C2

Specimen	Material property	Concrete f'_c, ksi	Steel plates F_y, ksi	M_y, kip.ft		M_{PSDM}, kip.ft		$\frac{M_{PSDM}}{M_y}$		$\frac{M_{exp}}{M_{PSDM}}$	
				Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
C1	Nominal	4.0	50.0	1649	-1440	3010	-2406	1.83	1.67	1.18	1.10
	Actual	4.5	55.4	1924	-1638	3387	-2640	1.76	1.61	1.04	1.00
	Expected	5.74	55.0	2300	-1529	3596	-2662	1.56	1.74	0.98	1.0
C2	Nominal	4.0	50.0	1627	-1411	3044	-2359	1.87	1.67	1.35	1.06
	Actual	5.1	55.4	2178	-1474	3639	-2623	1.67	1.78	1.13	0.96
	Expected	6.5	55.0	2371	-1520	3624	-2608	1.53	1.72	1.14	0.97

Figs. 3a and 3b shows a schematic of the damage on the steel plates for Specimens C1 and C2, respectively, at completion of the tests. In this figure, the locally buckled areas are marked with dashed lines, the fracture lines are shown with solid line, the failed tie welds are shown with dots and failed tie bar in Specimen C2 are shown by crosses. Note that failures at tie locations, in all observed cases, were due to weld failure at their connection (typically at only one end of the tie) except for one tie bar in Specimen C2 (identified by the red cross in Fig. 3b where the tie itself is known to have fractured). The failed end of the tie is marked in the figure.



(a)



(b)

Fig. 3 – Post-test damage inspection of the wall steel plates for: (a) Specimen C1, and; (b) Specimen C2

6. Summary and Conclusions

Two large-scale C-shaped Composite Plate Shear Wall/Concrete Filled were subjected to axial and cyclic flexural loading. The walls were subjected to 19% and 15% axial loading before horizontal cyclic loads were applied. Both specimens showed similar buckling and fracture behaviors. Cyclic ductility was more than 4 when flexural strength dropped to 80% of the peak value developed, and both specimens reached or exceeded their calculated plastic moment capacities in the positive and negative direction. Tests showed that even though local buckling started in early cycles after yielding, the capacity of the walls did not drop until fracture of the steel plates, and strength degradation was progressive and not sudden.

Based on the experimental observations and results obtained, it can be concluded that C-shaped C-SPW/CF can exhibit good cyclic behavior without premature strength degradation, while maintaining their ability to resist large axial load of up to 19% $A_c f'_c$.

7. Acknowledgments

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