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CYCLIC INELASTIC IN-PLANE BEHAVIOR OF CONCRETE FILLED STEEL SANDWICH PANEL WALLS

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ABSTRACT

The cyclic inelastic in-plane behavior of Concrete Filled Steel Sandwich Panel Walls (CFSSP-Walls), composed of double skin steel plates inter-connected through seamless tie-bars welded to the steel plates, was investigated through large-scale experiments and finite-element analysis. These walls are meant to be used where there is relatively large demand on the lateral load resisting system as they could offer ductile behavior while providing reduced cross section total thickness and fast constructability compared to other systems. Four large scale CFSSP-Wall specimens, with and without boundary elements, were tested under quasi-static cyclic loading to investigate the seismic behavior of this type of structural system. The tested walls had an aspect ratio (height to depth ratios) of roughly 2.5 and were able to develop their plastic moment capacity and undergo large drifts prior to buckling of the steel skin plate. Ultimate failure mode was fracture of the skin plates, starting from the tie-bar to steel plate interaction; yet, the tested specimens were able to sustain their flexural capacity up to drifts in excess of 3% and exhibited significantly ductile behavior.

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The cyclic inelastic in-plane behavior of Concrete Filled Steel Sandwich Panel Walls (CFSSP-Walls), composed of double skin steel plates inter-connected through seamless tie-bars welded to the steel plates, was investigated through large-scale experiments and finite-element analysis. These walls are meant to be used where there is relatively large demand on the lateral load resisting system as they could offer ductile behavior while providing reduced cross section total thickness and fast constructability compared to other systems. Four large scale CFSSP-Wall specimens, with and without boundary elements, were tested under quasi-static cyclic loading to investigate the seismic behavior of this type of structural system. The tested walls had an aspect ratio (height to depth ratios) of roughly 2.5 and were able to develop their plastic moment capacity and undergo large drifts prior to buckling of the steel skin plate. Ultimate failure mode was fracture of the skin plates, starting from the tie-bar to steel plate interaction; yet, the tested specimens were able to sustain their flexural capacity up to drifts in excess of 3% and exhibited significantly ductile behavior.

Introduction

CFSSP Sandwich walls are being considered by practicing engineers for construction in seismic regions of the USA, including as ductile flexural walls in high-rise applications. Their appeal is that they are envisioned to be highly ductile, redundant, of high strength, and easy and rapid to construct. Furthermore, use of a CFSSP Sandwich wall instead of a conventional reinforced concrete wall in building applications can translate into thinner walls with resulting greater leasable space. CFSSP Sandwich walls also have many of the same positive characteristics and advantages of other concrete-filled composite shapes, namely:

a) The steel can provide some confinement of the concrete and permit development of the full composite capacity, with stable seismic energy dissipation;
b) All the concrete in the member can contribute to strength and ductility of the member as concrete spalling does not occur;
c) The steel shell can act as formwork for the concrete;
d) Construction may be accelerated if the steel shell alone can provide resistance to dead-load;
e) The steel components can be fabricated off site in a controlled environment;

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f) The concrete core delays local buckling of the steel plate, and prevents its inward buckling, also enhancing ductility of the composite walls;
g) Reinforcing bars in the type of applications considered here are unnecessary in the concrete, and;
h) The final aesthetics can be architecturally pleasing.

Recent research by the authors has investigated the CFSSP Sandwich wall concept for seismic applications, by generating critical fundamental knowledge on its flexural cyclic inelastic and seismic behavior and performance. This project considered walls with and without boundary elements, and issues related to foundation design for proper load-transfer behavior. Laboratory experiments investigated the behavior of large-scale walls using quasi-static testing procedures. Nonlinear detailed finite element models are used to investigate local response mechanisms and expand the experimental results to include a wider range of parameters to define response. This on-going research work will serve as the basis to develop and validate design recommendations.

Wall Specimens

Two groups of two specimens were tested. In Group NB, the CFSSP Sandwich Walls have no boundary elements and their cross section is composed of double web skin plates having thickness, *t*, of 5/16" and width, *w*, of 40", connected through circular tie bars spaced equally in both horizontal and vertical direction at a spacing, *S*, that varies from one specimen to the other. The flanges of the panel consist of half HSS sections in order to avoid premature failure of the cross section's corner welds due to concentration of stresses that has been observed in prior research for rectangular sections (El-Bahey and Bruneau 2012a, 2012b). The tested specimens have a total width, *W*, of 48" (40" skin plates plus two half-HSS 8.625×0.325 of 8.625" diameter) and total thickness, *b*, of 8-5/8"). The height of both specimens in Group NB is 10 ft above the top of their footing, and their width is 4ft, resulting in an aspect ratio of 2.5. A typical cross-section of Group NB specimens is shown in Fig. 1.

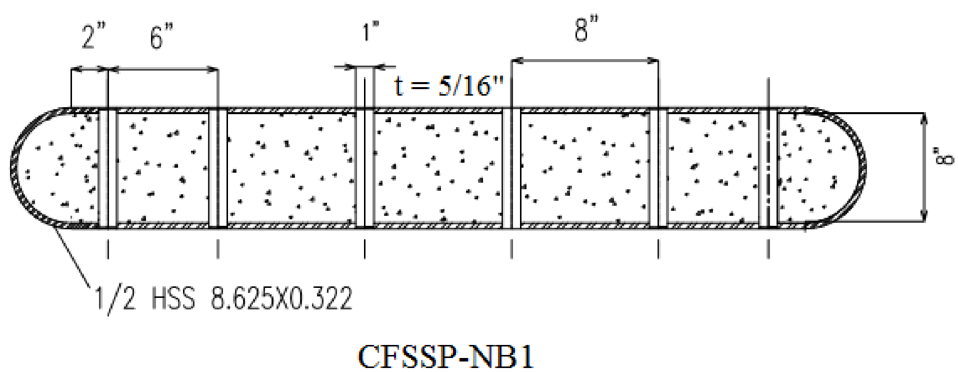

Figure 1. Representative Cross Sections for Group NB Specimens

Tie bars were welded to the web skin plate using plug welding: tie bars having a total length equal to *(b-t)*, were positioned to span the distance between the steel web plates center lines, and the remaining half thickness of the plate on each side was filled with welding material to create the plug weld. For specimen CFSSP-NB1, the spacing of the tie bars, *S*, in both horizontal and vertical direction was set to 8 inches, giving a flat width-to-thickness ratio, *S/t*, of 25.6. The spacing of the tie bars in specimen CFSSP-NB2 in both horizontal and vertical directions was 12 inches, for a corresponding ratio, *S/t*, of 38.4. The tie bars were designed to remain elastic throughout the tests, and consequently have a diameter of 1".

The HSS sections were cut using water jet in order to avoid inducing distortions due to residual stresses, and thus to avoid the need for annealing of the HSS after cutting. The half HSS section was welded to the steel web using full penetration welds.

Group B specimens were CFSSP-Walls having boundary elements (i.e., columns) consisting of concrete filled round HSS section. Their cross section were composed of HSS columns and of double web skin plates having width, *w*, of 30" and thickness, *t*, of 5/16" connected through tie bars spaced equally in both horizontal and vertical direction at a spacing, *S*, that varies from one specimen to the other. The two specimens in Group B had the same outer dimensions, plate thicknesses, and tie bar diameter. Specimen CFSSP-B1's tie bars were connected to the web plate through plug welding per the same technique used for the specimens of Group NB; tie bars were spaced at 8" center-to-center, leading to *S/t* ratio of 25.6 for the plate. However, for specimen CFSSP-B2, tie bars were assembled differently. Bars having total length of *(b+2t)* were used, *b* being the total width of the wall. As such, when installed, the tie bars protruded beyond the steel web plate on each side, and were fillet welded to the web steel plates. Tie bars for specimen CFSSP-B2 are spaced at 12" center-to-center, resulting in a *S/t* ratio of 38.4 for the plate. Fig. 2 shows the cross section of specimen CFSSP-B2.

The boundary elements used in Group B specimens were round HSS 8.625×0.322 welded to the steel webs using full penetration welding. Also, dimensions of the Group B specimens were chosen such that specimens in both groups B and NB would have approximately the same plastic moment capacity. Specified materials were ASTM A252 Gr 3 for the HSS, ASTM A572 Gr.50 for the plates, and 4 ksi self consolidating concrete for all specimens.

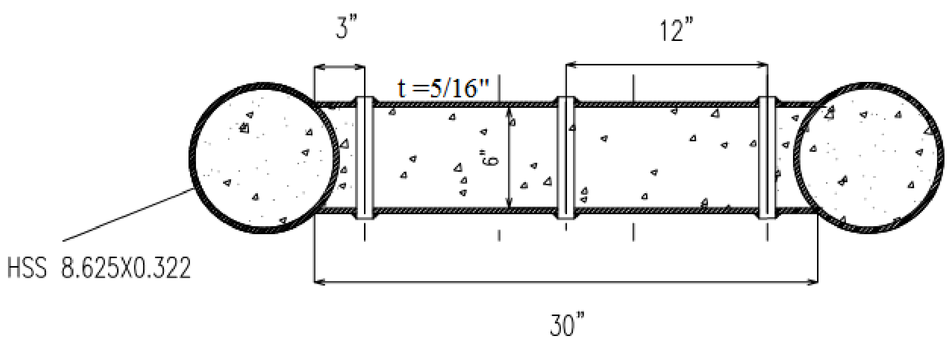

Figure 2. Cross Sections of Specimen CFSSP-B2

Test set-up

The test setup required design of a head plate at the top of the wall to transfer the actuator force to the tested specimen, a reinforced concrete foundation at its base to resist a moment of 3402 kip-ft, pre-tensioned threaded bars (Dywidags) to tie the reinforced concrete foundation to the lab strong floor, structural components transferring the design load from the tested CFSSP-Wall to the reinforced concrete foundation, and finally a side restraint system to prevent the tested wall from lateral motions during testing. Fig. 3 shows a typical elevation of this setup.

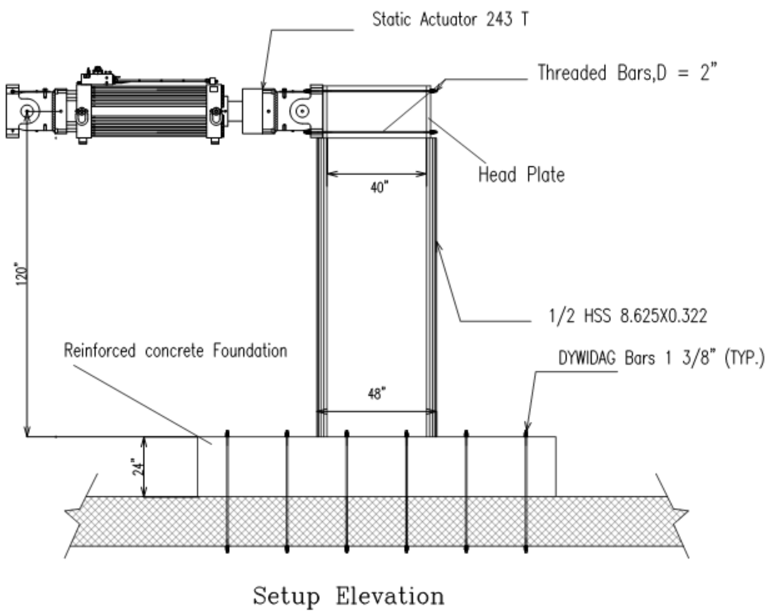

Figure 3. Test set-up

Cyclic Test Response

Only minor differences in response were observed among the 4 specimens. Observations are summarized here for specimen CFSSP-NB1, representative of typical behavior.

At 0.6% drift, strain gages on the specimen indicated the onset of yielding. Some flaking of white wash paint was noted, especially in parts of the HSS. There was no visual evidence of local buckling. At 1.2% drift the entire base of the wall seemed to be yielding, but there was still no observed local buckling.

Local buckling was first observed at 1.8% drift. This buckling was located on the web plate of the specimen, on one side of the wall, between the first and the second row of tie bars from the base. The half HSS part of the section did not buckle at this drift level. At 2.4% drift, the half HSS end of the cross-section developed local buckling at a location equivalent to between the first and the second row of ties above the footing, as shown in Fig. 4. Minor fractures appeared in some of the plug welds on the first row of tie bars.

At 3%, some of the tie bars plug welds fractured around the perimeter of the tie bars. During cycling, fractures also started to propagate from the circumference of the first tie bar in the first row towards the half HSS. The crack propagated by about 3/4 of an inch. At 3.6% drift, a first crack developed in the HSS part of the cross-section, on the face of the buckled wave in the half

HSS. Testing stopped after a few cycles at this 3.6% drift value, as the wall fractured over 18 inches of its base, in a plane passing through the center line of the first row of tie bars (Fig. 5).

Figure 4. The Local Buckling of the HSS Part at 2.4% Drift

Figure 5. Fracture of the CFSSP-NB1 Specimen East Side at 3.6% Drift

The hysteretic force-displacement curve obtained from this test is shown in Fig. 6. Results indicate that the wall exhibited ductile behavior up to significant drifts (considering that walls are stiff structural elements expected to develop relatively small drifts during an earthquake). Experimental results also indicated that all wall tested were able to develop their plastic moment computed using cross-section analysis and actual material properties. Such calculations will be provided in subsequent publications.

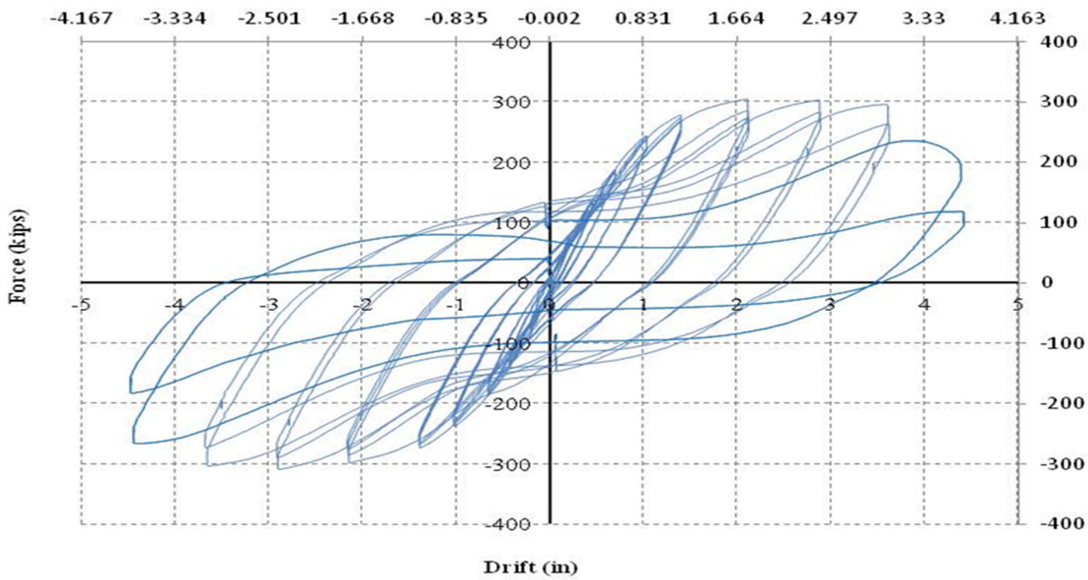

Figure 6. hysteretic force-displacement curve for Specimen CFSSP-NB1

Conclusions

Concrete-Filled Steel Plate Sandwich Walls can provide an alternative to regular reinforced concrete construction for seismic resistance. Four specimens were subjected to cyclic inelastic tests to investigate their cyclic inelastic behavior. Satisfactory behavior was obtained in all cases as large ductile flexural deformations were achieved before failure. Further studies using analytical and advanced finite element methods are underway to develop analytical and design tools for those systems, using models that have been validated against those experimental results.

Acknowledgments

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