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SEISMIC BEHAVIOR OF CONCRETE-FILLED STEEL SANDWICH WALLS AND CONCRETE-FILLED STEEL TUBE COLUMNS

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ABSTRACT

Concrete-Filled Steel Plate Sandwich Walls (CFSP Sandwich Walls) can provide a cost-effective alternative to regular reinforced concrete shear wall construction, with faster construction time. As a result, combined with its recognized multi-hazard resistant properties, this composite system has the potential to transform seismic design in many applications. However, in spite of all the foreseen excellent attributes of the structural system, lack of knowledge on its expected seismic performance has been an absolute impediment to its implementation. An analytical and experimental research program funded by the American Institute of Steel Construction has been conducted at the University at Buffalo to provide data on seismic response and generate design recommendations. The results of this research project, in parallel to research results on the multi-hazard performance of concrete-filled steel tubes, are summarized. Both systems are appealing as ductile flexural walls and columns in bridges or in high-rise applications.

Introduction

CFSP 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. Beyond accelerated construction for bridge applications and ductile seismic performance, use of a CFSP Sandwich wall instead of a conventional reinforced concrete wall in building applications can translate into thinner walls with resulting greater leasable space. CFSP 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;
(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 CFSP Sandwich wall concept for seismic applications, by generating critical fundamental knowledge on its flexural cyclic inelastic and seismic behavior and

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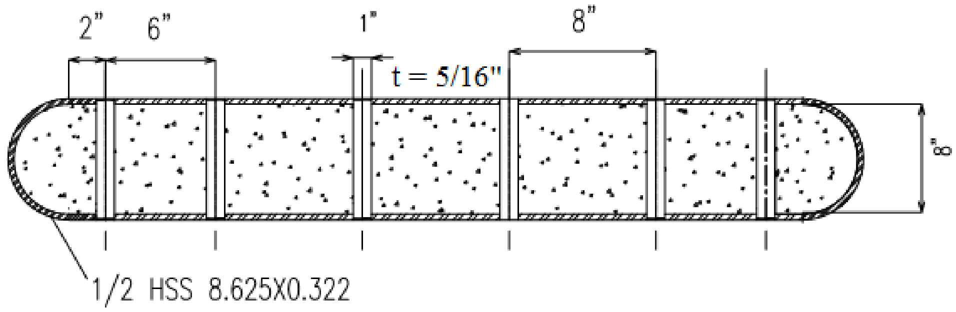
performance. This project considered walls with and without boundary elements, and issues related to foundation design for proper load-transfer behavior. Laboratory experiments investigated the behavior of large-scale walls using quasi-static testing procedures. Nonlinear detailed finite element models 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.

In parallel, research has been conducted on the multi-hazard performance of concrete-filled steel tubes. Some findings from that research are summarized here, as they provide insights on similarities on the seismic performance of various shapes within this class of concrete-filled members.

Wall Specimens

Two groups of two specimens were tested. In Group NB, the CFSP 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.



CFSSP-NB1

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, *b/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, *b/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 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 *b/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 *b/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.

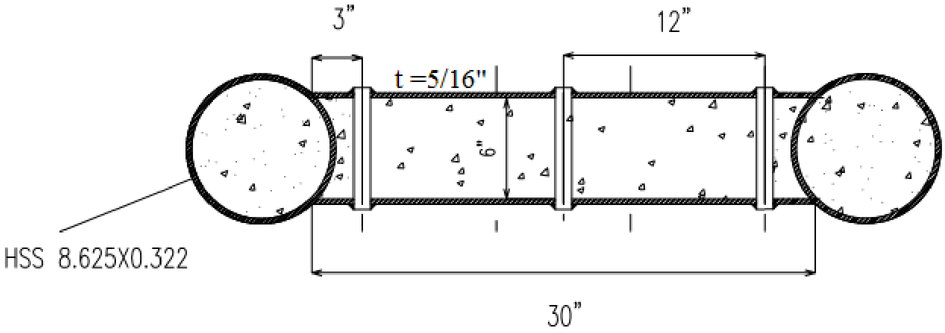


Figure 2: Cross Sections of Specimen CFSSP-B2

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.

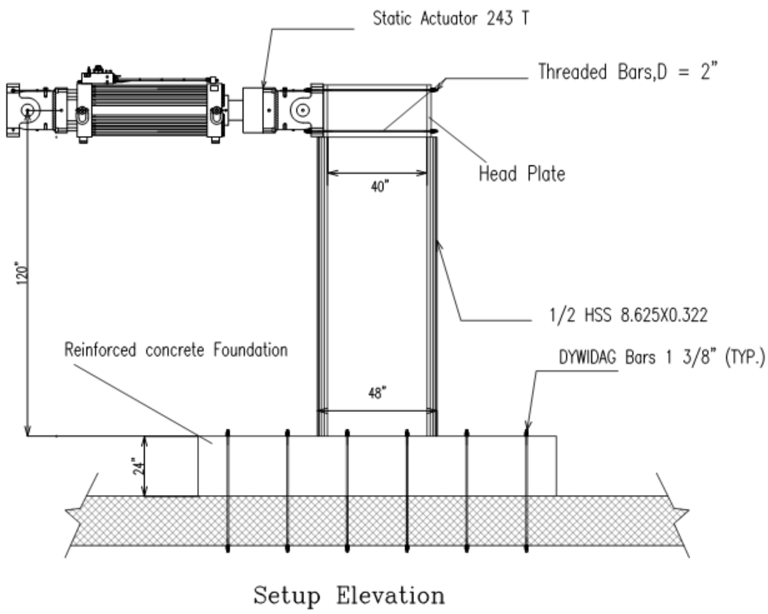


Figure 3: Test set-up

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).

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 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

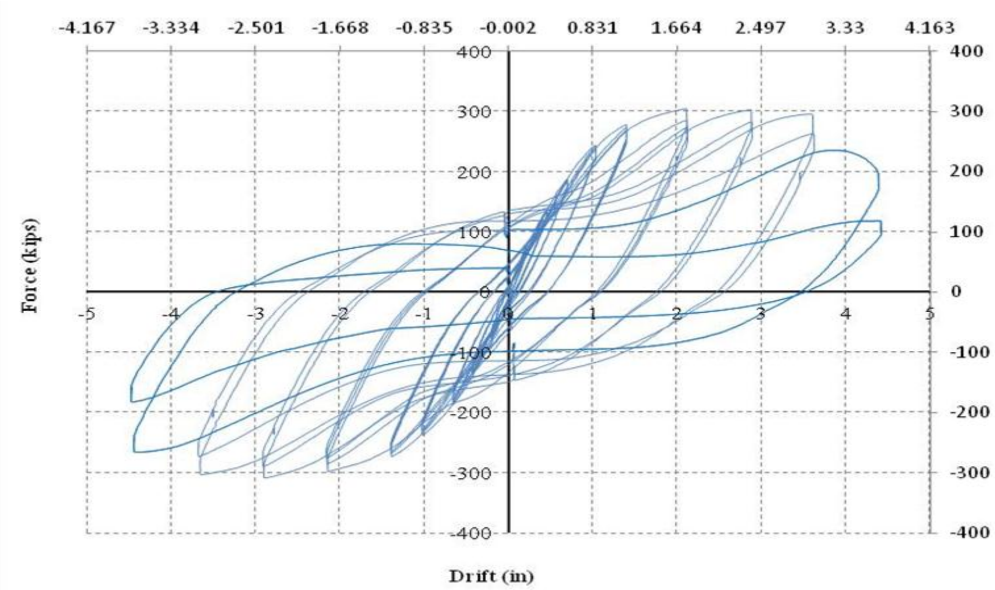


Figure 6: hysteretic force-displacement curve for Specimen CFSSP-NB1

Multi-hazard (Blast and Seismic) Resistant Concrete-Filled Columns

There is a growing concern that bridges piers, in addition to be resistant to earthquakes, must also be able to resist blast forces as a result of terrorism. In the USA, this concern is justified as threats have been received against bridges across the nation. In this context, highway bridges seem more accessible and vulnerable than landmark bridges which are closely monitored. In many instances, the destruction of a highway bridge can have profound effects on the economic circuit those infrastructures support. Protecting those critical components of the nation's infrastructure requires the development of single structural concepts that can provide multi-hazard protection. Such concepts should accommodate consistent demands imposed by the hazards and provide satisfactory performance for conflicting demands. Because of their inherent structural qualities, ranging from higher strength, substantial toughness and ductility and their applicability to situations requiring accelerated constructions or repairs, Concrete-Filled Steel Tubes (CFST), Concrete-Filled Double Skin Steel Tube (CFDST), and Modified Steel-Jacketed Columns (MSJC) are proposed as candidate concepts in that approach. Much of the multi-hazard research on CFST has been extensively published before (including literature reviews on behavior of CFST), and is not repeated here (see Marson and Bruneau 2004; Bruneau and Marson 2004; Fujikura et al 2007, 2008; Fujikura and Bruneau 2008, 2011a, 2011b).

Systems Description and Features

CFDST is a steel-concrete-steel “sandwich” section formed by two concentric steel tubes separated by a concrete core as shown on Fig. 7. That configuration seeks to draw upon the benefits in strength, toughness and stiffness derived for steel-sandwich construction by placing the steel at the periphery of a filler material (as described, for example, in Montague 1975). The outside skin at the periphery of the section provides strength and stiffness, the inside skin enhances ductility, and the concrete in between provides local and overall stability to the system. That synergy between the tubes and the core results in a more redundant section which completely fails only when all three layers of materials fail. In this section, concrete material failures such as spalling, breaching or scabbing under shock load are controlled. This helps preserve the stability of the section and guarantee enough (axial) residual strength for the section at ultimate conditions.

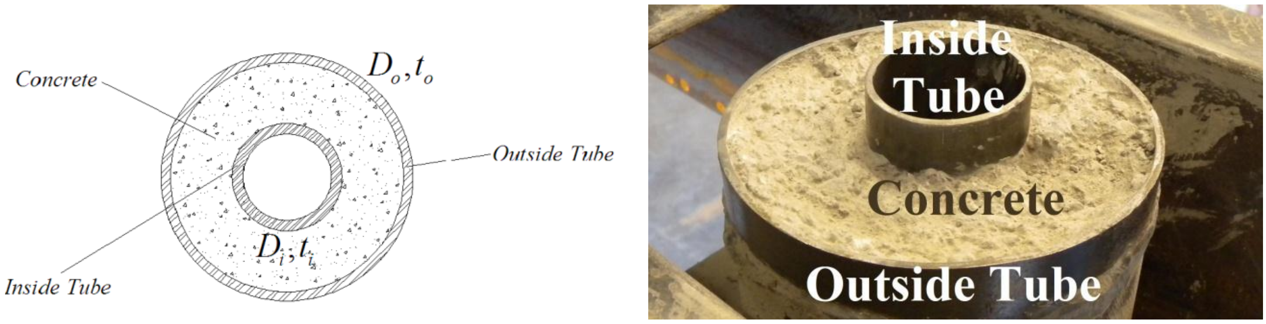


Figure 7. Concrete Filled Double Skin Tube

On the other hand, wrapping concrete column with a steel jacket is widely accepted as a cost effective retrofit technique for column of seismically deficient bridges. The merits of this technique do not translate however into improved blast performance for bridge column. Direct shear failure under blast load have been observed at the gaps between the jacket and the surrounding footing and cap beam when exposed to blast (Fujikura and Bruneau, 2008). By controlling this undesirable failure mode without hindering the initial role of the jacket, a multi-hazard resistant system is obtained. Structural steel collars placed around the gaps and tied to the adjacent elements with post-installed anchors can help increase the shear strength locally (Fig. 8Figure). A non-stick interface between the collar and the column can also be created to allow only smooth contact thus increasing only shear strength while leaving flexural strength of the column virtually unchanged as initially intended.


Figure 8. Steel Collar: Components (Left and Middle) and Construction (Right)

Blast and Seismic Tests

CFDST and MSJC column bents were tested under blast overpressures: The bents were laterally supported by a reaction frame that also serves to simulate the boundary condition and rigidity at the top of the bent beam that the deck of a full-scale bridge would have provided. Other CFDST column specimens were submitted to cyclic pushover to simulate earthquake excitations. The resulting designs yield sections that combined inside and outside tubes ranging from moderately ductile (MD) to highly ductile (HD). All specimens were quarter scale but a range of properties were taken into account. ASTM A513 steel tubes with a minimum yield of 290 MPa and self-consolidating concrete with a minimum strength of 34.5MPa were specified for the CFDST. The materials properties for the MSJC were the same as in Fujikura and Bruneau (2008). A 9.5mm A36 steel plate was used for the base of the collar, whereas the lip of the collar was cut from an A53 steel tube (216mm in diameter and 8mm-thick).

Measured plastic rotations after the blast tests were well in excess of 2 degrees for CFDSTs and MSJC (Figs. 9 and 10 respectively). For reinforced concrete columns, blast-induced plastic rotations of 2 to 4 degrees would lead to “moderate to heavy damage” (UFC, 2008) suggesting that repairs or complete replacement would be necessary. However, comparable test results for both CFDSTs and MSJCs showed that repair would be needed for aesthetical reasons only, and that the deformed columns would be able to perform their functions. For more severe blast charges, plastic rotations in excess of 4 degrees were accompanied with moderate to important local denting of CFDST section and seam failure of MSJC’s jacket. For near contact charge condition, fracture of the outside tube of the CFDST occurred, but the inner tube prevented complete failure of the columns.

Figure 9. Plastic Deformation of CFDST under Blast

Figure 10: Global (Left) and Local Deformation (Right) of MSJC after Blast Test

All specimen in the seismic test program exhibited ductile behavior up to failure. In all cases, yielding preceded buckling of the outside tube. Yielding first occurred at a drift level of about 1.5%, local buckling around at least 3%, and failure at well beyond 7.5%. Stiffness reduction was gradual, and no substantial reduction in strength was observed until failure. In general, hysteretic loops for the specimens were stable and exhibited pinching proportional to the severity of local buckling (Fig. 11Figure).

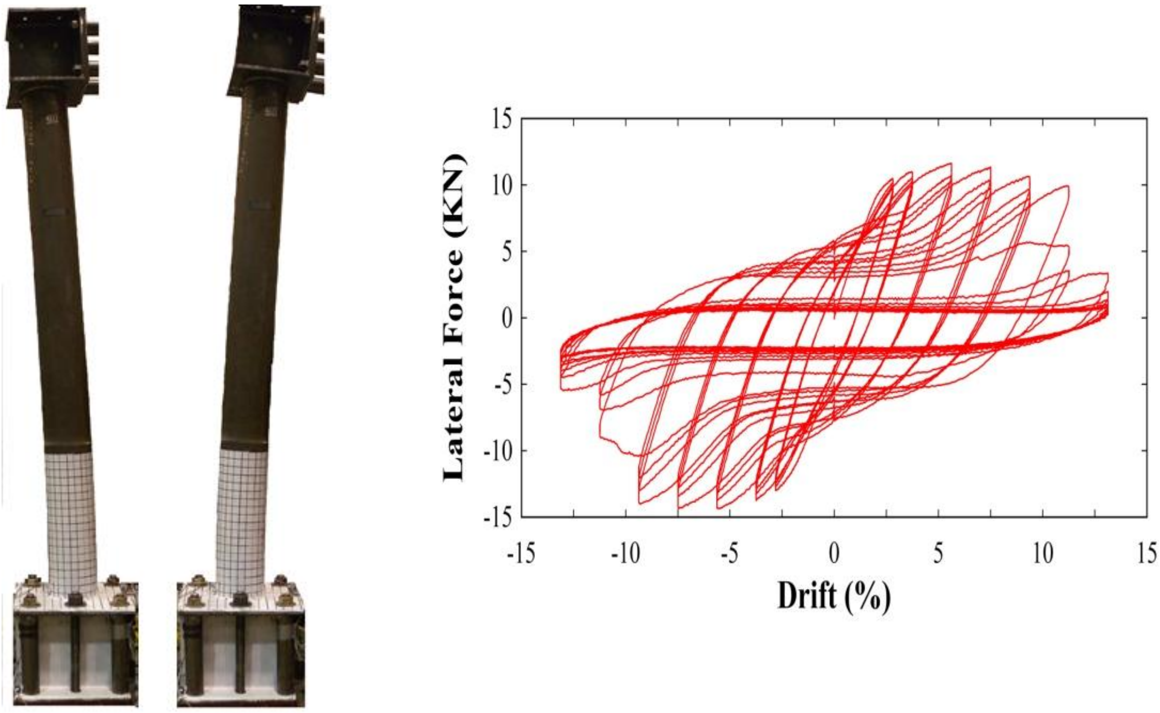


Figure 11: Specimen at Maximum Strength (Left) and Typical Hysteretic Loop (Right)

Conclusions

Concrete-Filled Steel Plate Sandwich Walls, as well as concrete-filled double skin steel tubes, can provide cost-effective alternatives to regular reinforced concrete construction, to provide seismic as well as blast resistance. Specimens were subjected to cyclic inelastic tests, as well as in some cases blast, to investigate their applicability as candidate multi-hazard systems for bridge applications. Satisfactory behavior was obtained in all cases. Large ductile flexural deformations were achievable during both seismic and blast tests.

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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