Development of Impulse-Damage Curves for Conventional Steel Stud Infill Walls

Bryan Bewick, Ph.D., P.E., Protection Engineering Consultants, 14144 Trautwein Road, Austin, TX, 78737, USA
Casey O'Laughlin, Jacobs Technology, 1020 Titan Court, Fort Walton Beach, FL, 32547, USA
Eric Williamson, Ph.D., P.E., The University of Texas at Austin, 1 University Station, Austin, TX, 78712, USA

INTRODUCTION

Cold-formed steel stud (CFSS) infill wall systems are an attractive building material for commercial and government facilities for several reasons. Cold-formed steel construction is cost-effective, and, as a standard construction method, architects and engineers have high confidence in their designs being built properly. Cold-formed steel allows for different types of aesthetics through façade alternatives and has good sustainability; it is recyclable and provides resistance to weather, insects, mold, and rot. CFSS systems also provide resistance in extreme environments such as blast loading because of the inherent ductility that steel possesses. Conventionally constructed CFSS systems are currently used for blast-resistant applications where they are subjected to relatively low blast loads. Past research, though limited, has shown that CFSS wall systems are also capable of resisting large magnitude blast loads when properly anchored at the supports [1, 2]. The high capacity CFSS systems resist large blast demands by having connections that are strong enough to allow for the vertical studs to develop their full tensile-membrane capacity. The use of specialized connections and construction methods, however, can be costly and the market for such applications is relatively small. Steel stud wall systems absorb energy with a large ultimate deflection, which limits the forces acting on the connections. However, because of a lack of data on the response of these types of systems when subjected to blast loads, current design guidelines for cold-formed steel walls are conservative. Currently, the stringent design requirements effectively exclude CFSS systems from mid-range load applications. More data is needed to support broader use of CFSS systems by relaxing the now somewhat restrictive response limits.

Accordingly, the Air Force Civil Engineering Center (AFCEC) has conducted research to enhance the knowledge-base for the blast resistance of cold-formed steel infill wall systems using conventional wall materials and construction practices. The most common configuration for CSSW is to have the vertical stud section connected to a track section with a 3.8 cm (1–½") flange dimension by means of a single self-tapping screw on either side of the wall section (2 screws total per stud connection at one end). However, it is somewhat common and allowable under the American Iron and Steel Institute (AISI) Standard Specification for the Design of Cold-Formed Steel Structural Members [3] to use a track with larger flange dimensions and additional screws. The main focus of the AFCEC program was to enhance the blast-resistant capacity of CSSWs using conventional construction techniques already utilized and allowed under the AISI codes. Track sections are available with up to 7.6 cm (3") flange lengths and under the AISI standard [3] with proper accompanying stud flange dimension it is possible to fit up to 6 screws into a single stud-to-track connection (12 total screws and each end where a stud terminates into a track section). Figure 1 illustrates a stud connected to a 7.6 cm (3") track using 3 screws spaced as allowed by the AISI codes. In previous research at AFCEC, an extensive experimental program focused on the effects of common CFSS wall system materials and construction practices on blast resistance [4, 5]. The results of the research showed that conventional CFSS systems have higher capacities than the current design criteria. Furthermore, with enhancements to the CFSS systems using details that are still considered conventional practice and allowed by AISI codes, even higher levels of protection (LOPs) are able to be attained by conventional CFSS designs with small impacts to the construction costs.

Figure 1. Conventional stud connection into a 7.62 cm (3”) track with 3 screws.

The research presented here had two major objectives; 1) to define LOPs for a blast-tested set of CFSS designs, and 2) characterize the enhanced response of CFSS walls with improved connections to determine if the UFC prescribed response limits are overly restrictive [6, 7]. The results of the analyses were used to support decisions regarding minimum conventional construction standoff distances as defined in UFC 04-010-01 [7]. The configurations tested/analyzed ranged from standard conventional designs to slightly enhanced configurations. The enhancements are all allowed under the current AISI design guidance [3] and are representative of systems that can be tweaked with better connections. The results also allow for a set of prescriptive designs that can be implemented at a lower than many of the commonly detailed blast designed CFSS walls. AFCEC’s previous research has been leveraged for the current effort, and eight CFSS wall systems have been investigated in detail by Protection Engineering Consultants (PEC) using finite element (FE) analyses to determine the relationship between wall damage and the impulse applied to the wall system for a given blast load scenario.

APPROACH

A series of computational models was used to determine the minimum standoff that each design could resist specified blast threats using the charge weights specified by UFC 04-010-01 [7]. This was done by refining an FE modeling approach that was validated against blast experiments. The validated models were then used to iterate on varying blast demands to determine damage and impulse relationships.

PEC used results of seven CFSS walls that were tested in full-scale arena blast tests over the course of two experiments by AFCEC in 2011 and 2012 [8]. The designs were tested against the charge weight 1 (CW1) from the now outdated UFC 4-010-01 from 2007 [9]. The FE methodology was extended to also look at 5 other CFSS designs against a smaller sample of blast demands. The scope of the walls which were analyzed is outlined in Table 1. Design considerations that were considered and varied in the designs include stud track and thickness, stud flange width, track flange width, screw connections, deflection connections, and veneers (brick and EIFS facades). For SSW2 only, an interior steel sheet was placed on the interior between the vertical studs and interior gypsum sheathing.

Table 1. Overview of walls analyzed.

Bay ID	Size of Wall	Track ID	Stud ID	Stud to Track Connection	Stud Spacing	Anchor Spacing	Anchor Type	Interior Sheathing	Exterior Sheathing
SSW1	12ft x 17ft	600T300-54 (16ga)	600S162-54 (16ga)	3 screws	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, EFIS Stucco
SSW2	10ft x 10ft	600T300-54 (16ga)	600S162-54 (16ga)	3 screws	16 in.	16 in.	½" Titen HD	18ga Sheet Steel, Gypsum Board	OSB, Vapor Barrier, Brick
SSW3	10ft x 10ft	600T300-43 (18ga)	600S162-43 (18ga)	3 screws	12 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, EFIS Stucco
SSW4	10ft x 10ft	600T300-43 (18ga)	600S162-33 (20ga)	3 screws	16 in.	16 in.	½" Titen HD	18ga Sheet Steel, Gypsum Board	OSB, Vapor Barrier, Brick
SSW5	10ft x 10ft	600T300-43 (18ga)	600S137-33 (20ga)	Dietrich deflection clip	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, Brick
SSW6	10ft x 10ft	600T300-97 (12ga)	600S200-97 (12ga)	6 screws	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, EFIS Stucco
SSW7	10ft x 10ft	600T150-33 (20ga)	600S137-33 (20ga)	1 screw	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, Brick
SSW8	10ft x 10ft	600T150-33 (20ga)	600S137-33 (20ga)	1 screw	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, EFIS Stucco
SSW9	10ft x 10ft	600T150-54 (16ga)	600S137-54 (16ga)	1 screw	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, Brick
SSW10	10ft x 10ft	600T150-54 (16ga)	600S137-54 (16ga)	1 screw	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, EFIS Stucco
SSW11	10ft x 10ft	600T150-54 (16ga)	600S137-54 (16ga)	Double track de-flection connection	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, Brick
SSW12	10ft x 10ft	600T150-54 (16ga)	600S137-54 (16ga)	Double track de-flection connection	16 in.	16 in.	½" Titen HD	Gypsum Board	OSB, Vapor Barrier, EFIS Stucco

There are two aspects to the approach of the effort; the FE modeling approach, and the determination of damage for the CFSS walls.

FE Approach

The experiment data [8] was used to refine an FE modeling approach for CFSS walls. The FE modeling was completed using the commercial FE code LS-DYNA [10] to perform explicit dynamic analyses. The approach was built upon prior research efforts [5]. Most parts were modeled using shell elements. However, brick and mortar elements were modeled using solid elements as illustrated in Figure 2-a.

Steel stud materials were modeled using the piecewise linear plasticity material model. An implicit simulation of a coupon tensile test was used to develop the appropriate true stress-strain values to be implemented into the material model based on available engineering stress-strain values. Screw connections (stud-to-track, stud-to-gypsum, and stud-OSB) were modeled using discrete beam elements that applied the *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM material model calibrated to laboratory test data [4, 5]. The OSB sheathing was modeled using the *MAT_COMPOSITE_FAILURE_SHELL_MODEL material model with values based upon research of composite wood panels subjected to impact loads [11]. The gypsum material was modeled using the piecewise linear plasticity model calibrated to laboratory 3-point bending experiments [12]. The brick and mortar were modeled using the *MAT_CSCM_CONCRETE option. Anchors were modeled using the *MAT_PLASTIC_KINEMATIC material model. A spotweld definition was applied to the anchors to simulate the capacity of the anchor mounted in concrete, which allows for anchor failures where the concrete is failed as opposed to failure of the anchor material itself; illustrated in Figure 2-b. EIFS does not contribute significantly to the structural capacity of the system, so the EIFS veneers were implemented in the models by adding extra mass to the exterior OSB sheathing parts.

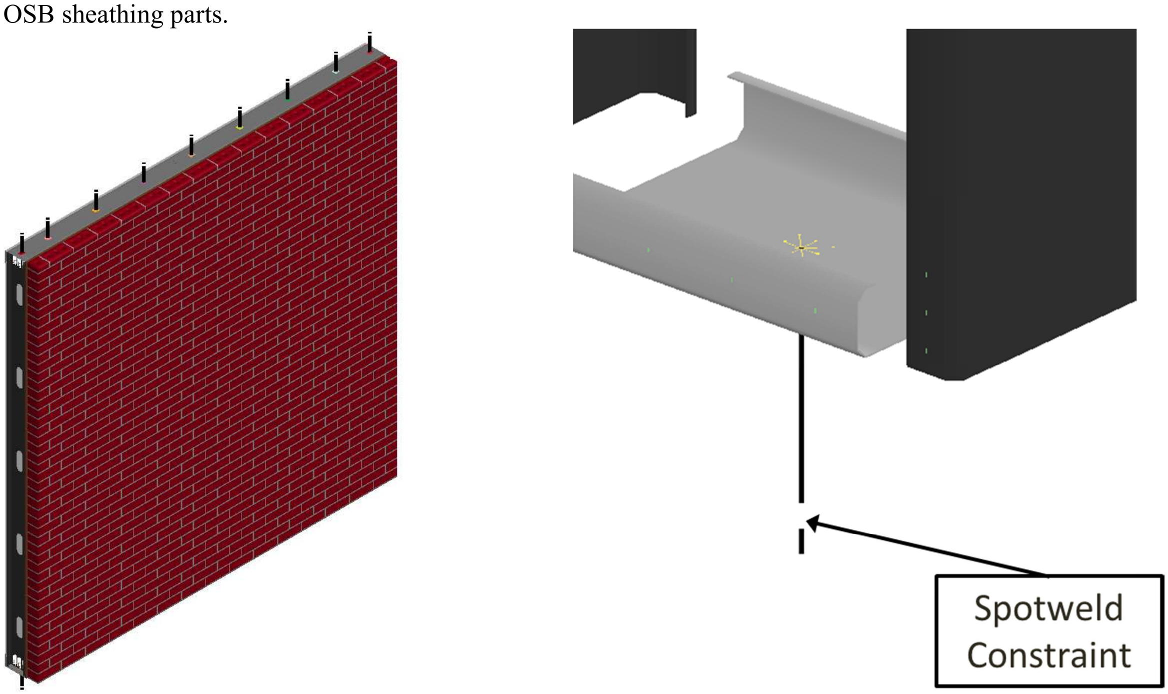

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Figure 2. Model setup a) overall model view and b) anchor treatment

For the validation cases, loads were applied based on gauge readings from the blast arena experiments. For the iterative cases used to develop LOPs, the *LOAD_BLAST_ENHANCED keyword was used to apply the blast loads. Since the negative phase pressures are most often not taken into account for a typical blast design analysis, the applied loads were defined with no negative phase included. Contacts were defined using the robust two-way automatic surface to surface contact definition.

Damage Approach

To determine a LOP for the modeled CFSS designs; two sets of criteria were applied. An LOP was defined for component damage and an LOP was defined based on injury hazard to building inhabitants from failed gypsum and stud wall components that become debris inside the structure. The overall LOP is governed by the lowest LOP as defined by the two sets of criteria; component damage and injury hazard.

LOP based on system and component damage have been defined by the US Army Corps of Engineers in PDC-TR 06-08 [6]. PDC-TR 06-08 provides qualitative descriptions of system and component damage for varying levels of protection. For this effort, the qualitative descriptions have been altered to be specific to the types of responses seen in conventional CFSS wall designs. The descriptions are summarized in Table 2. The descriptions are based largely on the behavior of the vertical studs. The damage level is correlated through whether the vertical stud has had no damage, compression flange buckling (CFB), CFB and stud rotation, or CFB, stud rotation, and connection failure. Gypsum failures are considered separately under the injury hazard definitions.

Table 2. Structural and component damage associated with Level of Protection

LOP	Description of Damage

Below AT Standards	Vertical stud connection failure with significant inward velocity.
Very Low (VLOP)	Damage to stud-to-track connections or right at failure.
Low (LLOP)	Compression flange buckling and stud rotation (tension flange buckling may also occur, but has not been found to be a consistent occurrence)
Medium (MLOP)	Compression flange buckling without stud rotation.
High (HLOP)	No stud buckling.

The system and component damage is illustrated in Figures 3 and 4. For HLOP, the vertical studs remain elastic during the blast event with no permanent plastic deformation to the system (Figure 3-a). At the MLOP threshold, there is enough stress in the vertical studs that they deform plastically. The plastic deformation creates a hinge at midspan. This hinge is generated though flange buckling of the vertical stud. The buckling always occurs on the compression flange (exterior flange), and often also occurs on the tensile flange but not always (Figure 3-b). The mechanism that was observed for walls that exhibited the MLOP response was that the flanges would buckle, but the interior and exterior sheathing were still connected which meant that there was no stud rotation occurring. At the LLOP threshold, the vertical studs would buckle and there would be stud rotation (Figure 4-a). The stud rotation is a mechanism that would occur most often when the interior gypsum sheathing becomes detached and the stud is no longer braced against rotation. As with the MLOP cases, there is always compression flange buckling. Tension flange buckling often occurred, but was not consistent. There were cases where the gypsum detached early enough in the response that the compression flange buckled and the stud rotated out of plane before the stresses could build enough in the tension flange to generate buckling. The VLOP criterion was similar to the LLOP criterion except that in the VLOP responses there was damage that occurred to at least one of the connections. In some instances this would be a single screw failure, in other instances there would be several connection failures. However, the majority of instances where a VLOP level was determined, the LOPs were based on injury hazard definition of failed gypsum panels (Figure 4-b). If all the connections on either the top or the bottom (or all connections) failed, this was considered to be below AT Standards. The responses that fall into this category are substantial failures that induce large velocities of the system into the interior of the structure.

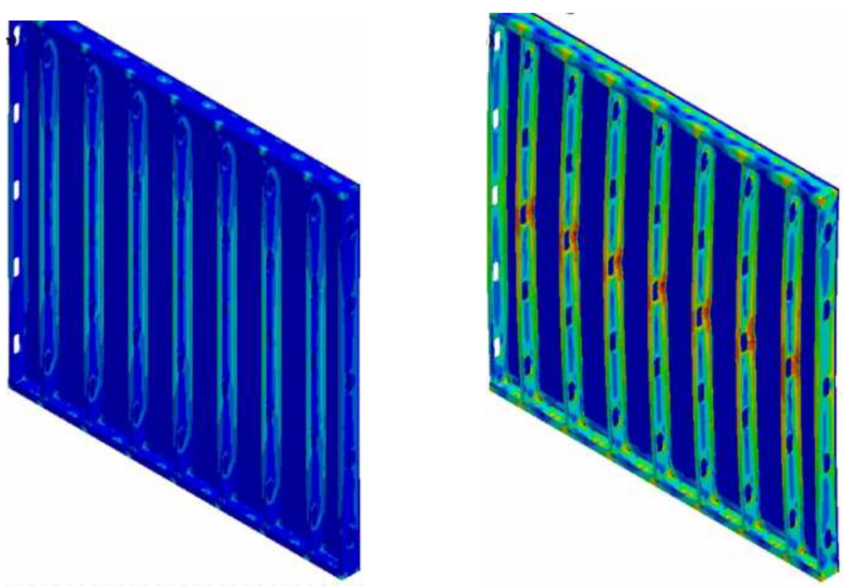

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Figure 3. Illustration of a) HLOP and b) MLOP damage-based responses

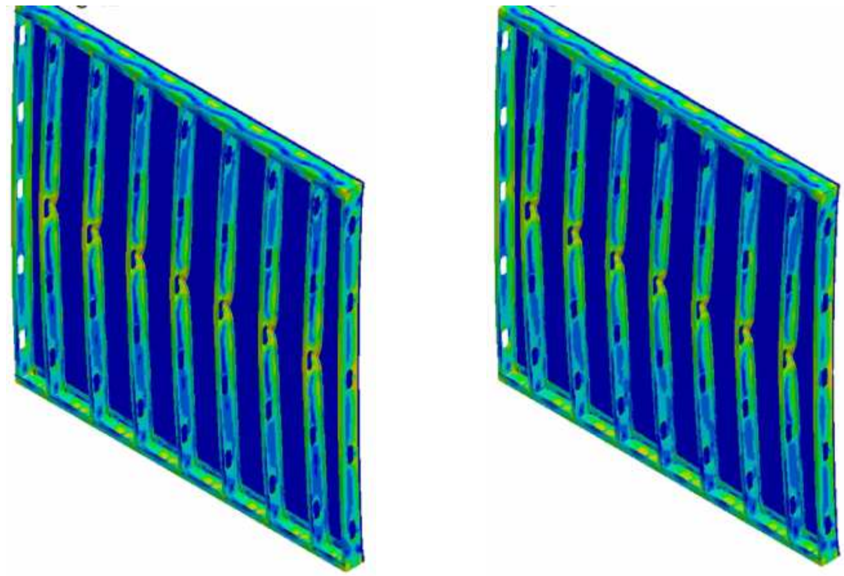

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Figure 4. Illustration of a) LLOP and b) VLOP damage-based responses

The above listed LOP definitions are based on the remaining structural integrity of the CFSS structural components. However, the LOP based on injury hazards can be considered as well. The LOP based on injury hazard is based on the energy imparted to inhabitants through disconnected gypsum panels that fly into the interior of a structure. There are fewer LOPs defined for the injury hazard cases. It is based on previous research for fragment impacts where 79 joules was presented as an accepted criterion for determining debris hazard based on mass and inward velocity of fragments [13]. The 79 joule criterion was based on small dense fragments as opposed to larger surface area components such as gypsum panels. However, this comparison provides a useful qualitative comparison for limiting LOPs based on injury criteria. For a majority of the cases, the gypsum came off in large pieces (except for cases with very stiff walls such as 12 Ga. studs). Thus, the assumption is conservatively that two 122 cm x 244 cm (4' x 8') fastened in a horizontal configuration will impact an inhabitant. The application of the injury hazard criterion was applied as described in Table 3.

Table 3. Application of injury hazard criterion

LOP	Description of Damage
Below AT Standards	Substantial inward velocity well above the 79 J cutoff, Severe Life Threatening
Very Low (VLOP)	KE >79 J within 10%, severe/life threatening
Low (LLOP)	KE is between 0 and 79 J, Minor to moderate
High (HLOP)	Gypsum remains attached to the stud wall system.

The overall LOP was determined by the lowest LOP according to the two types of LOP criteria. Most often, the VLOP walls were governed by the inward velocity of failed gypsum panels as opposed to the loss of connections in the stud wall.

VALIDATION

The seven test data points were used to refine the modeling approach. The approach was consistently applied over all seven cases and errors were all within less than 10%, with the FE models erring on the conservative side (higher predicted deflections than test data). For brevity, only the results for SSW3 (see Table 1) are reported here. SSW3 is a CFSS wall with 18 ga (0.11 cm thick) studs and track. The stud flange is 4.12 cm (1.62”) wide and the track flange is 7.6 cm (3”) tall. 3 screws have been used at each stud-to-track connection. There is an EIFS veneer layered on top of an OSB exterior sheathing. On the interior, there is gypsum sheathing. Figure 3 illustrates a typical response overlay with the test data. The wall was subjected to the CW1 blast threat from the now outdated 2003 version of UFC 4-010-01 [14]. The results tend to match very well on the inbound response. The peak deflections of the FE models are typical 10% above the test peak deflections. The FE models tend to rebound slower than the test walls. But overall the simulations match the test data quite well. The approach was deemed appropriate and used to populate synthetic test results for responses of the CFSS systems to varying blast demands.

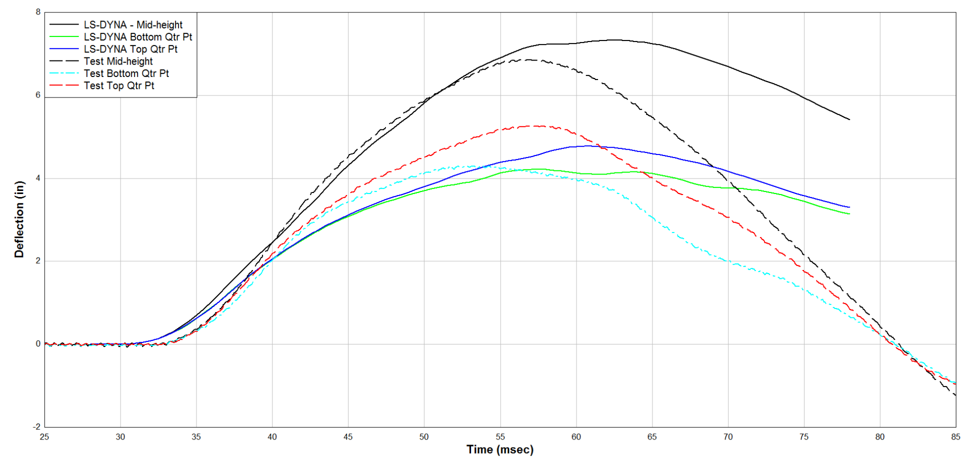
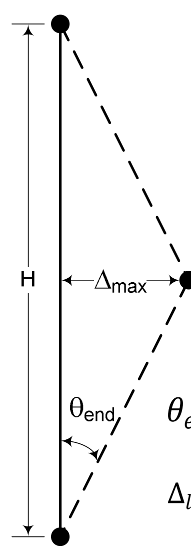


Figure 5. Typical overlay of FE validation against test data

APPLICATION OF APPROCH TO DEFINE LOP

For each of the designs, an iterative procedure was used where two threats (CW1 and CW2 from UFC 4-010-01) were applied for varying standoffs. The results were assembled into plots that show the effects of the applied loading to the deflection and rotation. The deflections were taken from nodal histories on the face of the interior flange similar to how data would be collected in full-scale blast tests. The rotations were calculated based on an assumption that the stud hinges at midspan. The HLOP responses do not have any hinging so the calculated rotations have some inherent errors for walls with HLOP responses. The rotation calculation is based on the response as depicted in Figure 6.



$$\theta_{end} \cong \frac{2\Delta_{max}}{H}$$

$$\Delta_{limit} = \frac{\theta_{end}}{2} \times 100$$

Figure 6. Three hinge response mechanism

Several design variances were considered in the effort as described in the approach. The three main considerations that determined the response type were the connection type, the stiffness of the stud system, and the type of façade used (brick vs. EIFS). The connection type (number of screws or deflection connections) did not greatly affect the response mechanisms (i.e. hinging or stud rotation), but rather impacted the ultimate attainable deflection. The stiffness of the system often changed how the LOP was determined. For instance, less stiff systems would respond slower and often the structural integrity (hinging, stud rotation, connection failures) would govern the LOP determination. On the other hand stiffer systems, such as the SSW6 design (see Table 1), have very high capacities that are often limited by the connection design. The SSW6 connection design was substantial enough that the LOP for the SSW6 design was governed mostly by the injury hazard criteria. The stiffer studs move quickly and fail the gypsum early in the event which provides for a substantial inward velocity that makes the wall rate at lower protection levels before there is significant damage to the structural components.

The effect of the exterior façade provides the largest difference in the CFSS system responses, however. It affects the response in two ways. Firstly, and most obvious, the added mass from a brick façade in comparison to the EIFS façade greatly increases the blast demand that a system can withstand. More energy is absorbed through inertia which means that the stresses in the vertical studs are lower. The other factor is that the rate at which the peak midspan deflection increases as the applied impulse increases is lower for walls with brick façade. There are two factors as to why this is the case. One reason is again the added mass. Walls with brick facades have lower inward velocities and are stressed less. The second reason is that since the inward velocity is lower, the interior sheathing is less likely to detach. While there is not much structural capacity to be had from the gypsum itself, the fact that the studs remain braced and not able to rotate about their weak axis means that the applied impulse is resisted better as opposed to an un-braced section that rotates.

To illustrate the discussion points above, a comparison of two walls with brick facades and two walls with EIFS facades are presented. The results of the analyses were plotted in terms of applied impulse and the corresponding peak deflection/rotation. The impulses have been normalized against the UFC 4-010-01 LLOP threat. Figure 7 illustrates a comparison of the responses and corresponding LOPs for two walls with brick facades. SSW2 (Figure 7-1) has interior sheet steel sheathing. The wall is well-braced and does not rotate out of plane. As a result, the response behavior is fairly uniform up to the point that the load begins to overwhelm the connections and there is debris hazard from the interior gypsum sheathing. SSW5 (Figure 7-b) is the other extreme as far as capacity of the system. SSW5 has a commercial deflection clip with 3 screws that attach the stud web. The wall has about 50% higher deflections as compared to SSW2 due to the ability for the vertical studs to move more in the vertical connection before fully loading the screw connections. As with SSW2, the wall does not rotate very much. This is due to the slow inward velocities which do not fail the gypsum connections and brace against weak axis rotation. The stronger wall (SSW2) is able to withstand applied impulses which are higher before failures occur.

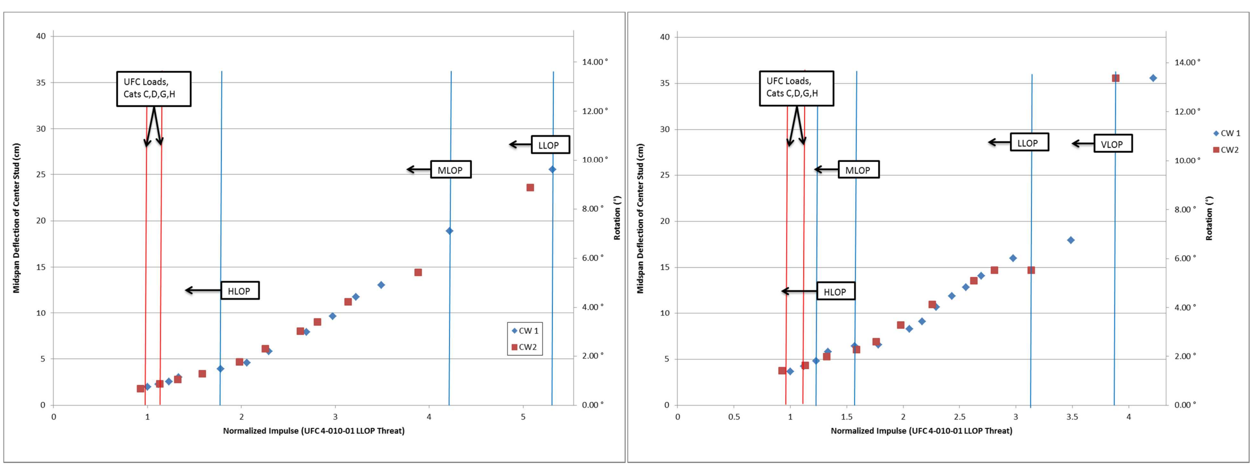


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Figure 7. Response curves for a) SSW2 and b) SSW5

The typical response for the walls with EIFS façade is a bit different from the walls with brick façade. The lighter systems have higher inward velocities. Figure 8 shows the responses for the two extremes, SSW8 (Figure 8-a) was the weakest specimen and SSW6 (Figure 8-b) was the strongest specimen. Figure 8-b shows a response that was typical of most EIFS façade walls. There is a linear relationship between the applied impulse and the peak midspan deflection up to the HLOP threshold. After the HLOP threshold, the slope is steeper (the secondary slope on the brick façade walls was only slightly steeper, refer to Figure 7). Once the vertical studs buckle, there are higher deflections because there is less mass to help slow down the inward velocity. Also, due to the higher velocities compared to brick façade walls, the gypsum interior sheathing detaches and breaks up more frequently. Without the interior sheathing there to act as bracing the EIFS façade walls turn on their weak axis more readily as compared to walls with brick facades. The weaker walls (SSW8; Figure 8-a) tend to buckle at very low loads. The SSW8 wall had 1 screw per connection point. Because of the large inward deflections, there were more stresses applied to the connections, and failures occurred at lower impulses as compared to SSW6. SSW6 had capacity to applied impulses that are 10 times the UFC 4-010-01 LLOP threat level, whereas SSW8 was able to sustain a protection level only to about 3.5 times the UFC 4-010-01 LLOP threat level.

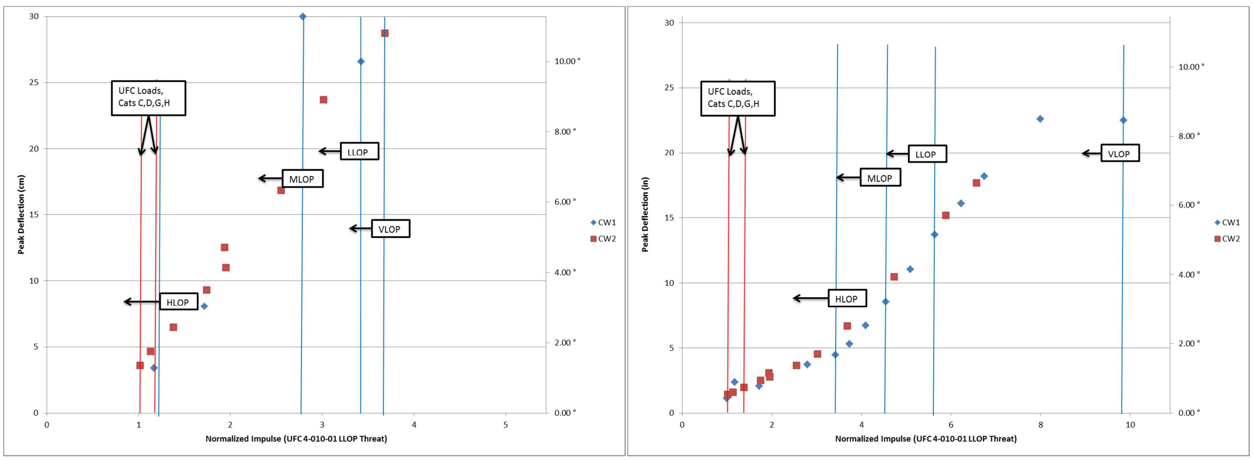

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Figure 8. Response curves for a) SSW8 and b) SSW6

CONCLUSION/SUMMARY/RECOMMENDATIONS

Detailed FE analyses were performed to develop damage limitations for a set of steel stud designs. The result was a set of prescriptive designs that can be implemented for a range of blast demands. The results also demonstrate the findings from previous research that slight modifications to conventional construction techniques provide higher capacities for CFSS systems. The analyses exhibit the resiliency of the CFSS systems when compared to the current design limits. The designs/configurations are capable of larger deflections with minor damage compared to the current UFC response limits [6, 7]. The designs/configurations also were able to withstand impulse loadings that are up to 4 times the UFC design loads [7].

Current UFC standoff distance and response limits both have high levels of conservatism built into them. Conventional designs are capable of withstanding higher prescribed loads and are capable of having larger midspan deflections than currently allowed under the UFC criteria. The analyses also show that with enhancements to conventional designs that are already allowed under the standard design codes, CFSS systems are capable of resisting higher impulse loadings as well as having higher midspan deflections than currently allowed.

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