AFRL-RX-TY-TR-2009-4600

EVALUATION OF SHEAR TIE CONNECTORS FOR USE IN INSULATED CONCRETE SANDWICH PANELS

Clay J. Naito
Department of Civil and Environmental Engineering
Lehigh University, PA 18015

John M. Hoemann
Applied Research Associates
P.O. Box 40128
Tyndall Air Force Base, FL 32403

Bryan T. Bewick and Michael I. Hammons
Air Force Research Laboratory

DECEMBER 2009

Interim Report for 1 January 2009 – 30 November 2009

DISTRIBUTION STATEMENT A: Approved for public release; distribution unlimited.

AIRBASE TECHNOLOGIES DIVISION
MATERIALS AND MANUFACTURING DIRECTORATE
AIR FORCE RESEARCH LABORATORY
AIR FORCE MATERIEL COMMAND
139 BARNES DRIVE, SUITE 2
TYNDALL AIR FORCE BASE, FL 32403-5323

NOTICE AND SIGNATURE PAGE

Using Government drawings, specifications, or other data included in this document for any purpose other than Government procurement does not in any way obligate the U.S. Government. The fact that the Government formulated or supplied the drawings, specifications, or other data does not license the holder or any other person or corporation; or convey any rights or permission to manufacture, use, or sell any patented invention that may relate to them.

This report was cleared for public release by the Air Force Research Laboratory Airbase Technologies Division Public Affairs Office and is available to the general public, including foreign nationals. Copies may be obtained from the Defense Technical Information Center (DTIC) (http://www.dtic.mil).

AFRL-RX-TY-TR-2009-4600 HAS BEEN REVIEWED AND IS APPROVED FOR PUBLICATION IN ACCORDANCE WITH ASSIGNED DISTRIBUTION STATEMENT.

__//signature//________________________
PAUL E. SHEPPARD
Work Unit Manager

___//signature//___________________
JOHN S. MASCELLI, Captain, USAF
Acting Chief, Force Protection Branch

___//signature//_______________________
ALBERT N. RHODES, PhD
Acting Chief, Airbase Technologies Division

This report is published in the interest of scientific and technical information exchange, and its publication does not constitute the Government's approval or disapproval of its ideas or findings.

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*	2. REPORT TYPE	3. DATES COVERED *(From - To)*
01-DEC-2009	Interim Technical Report	01-JAN-2009 -- 30-NOV-2009

4. TITLE AND SUBTITLE
Evaluation of Shear Tie Connectors for Use in Insulated Concrete Sandwich Panels

5a. CONTRACT NUMBER FA4918-07-D-0001
5b. GRANT NUMBER
5c. PROGRAM ELEMENT NUMBER 62102F
5d. PROJECT NUMBER 4915
5e. TASK NUMBER F2
5f. WORK UNIT NUMBER Q210FA72

6. AUTHOR(S)
+Naito, Clay J.; *Hoemann, John M.; **Bewick, Bryan T., **Michael I. Hammons

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
+Department of Civil and Environmental Engineering, Lehigh University, PA 18015
*Applied Research Associates, P.O. Box 40128, Tyndall AFB, FL 32403

8. PERFORMING ORGANIZATION REPORT NUMBER

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
**Air Force Research Laboratory
Materials and Manufacturing Directorate
Airbase Technologies Division
139 Barnes Drive, Suite 2
Tyndall Air Force Base, FL 32403-5323

10. SPONSOR/MONITOR'S ACRONYM(S) AFRL/RXQF

11. SPONSOR/MONITOR'S REPORT NUMBER(S) AFRL-RX-TY-TR-2009-4600_

12. DISTRIBUTION/AVAILABILITY STATEMENT
Distribution Statement A: Approved for public release; distribution unlimited.

13. SUPPLEMENTARY NOTES
Ref AFRL/RXQ Public Affairs Case #10-023. Document contains color images.

14. ABSTRACT
Protection against blast loads has become a high priority for many government agencies. Blast retrofitting and structural hardening, much like earthquake retrofits, can prove to be costly. For this reason, it is important to understand that any structural element has an inherent capacity to absorb energy and provide some level of protection. An evaluation of an existing structure may allow a designer to utilize the full absorption capacity of a structural element, precluding the need for a blast-specific retrofit. To illustrate this concept, the blast resistances of non-load bearing precast/prestressed or tilt-up concrete sandwich wall panels (WP) are examined. These components are used extensively in modern construction for cladding of framed building systems and often provide a significant level of protection from blast events.

15. SUBJECT TERMS
sandwich wall panels, prestress wall panels, precast wall panels, tilt-up wall panels, concrete insulted wall systems

16. SECURITY CLASSIFICATION OF: a. REPORT	b. ABSTRACT	c. THIS PAGE	17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
U	U	U	UU	37	Paul Sheppard
					19b. TELEPHONE NUMBER *(Include area code)*

Reset

Table of Contents

List of Figures

List of Tables

Summary

Protection against blast loads has become a high priority for many government agencies. Blast retrofitting and structural hardening can prove to be costly. It is important to understand that any structural element has an inherent capacity to absorb energy and provide some level of protection. An evaluation of an existing structure may allow a designer to utilize the full energy absorption capacity of a structural element precluding the need for a blast-specific retrofit. To illustrate this concept, the blast resistances of non-load bearing precast/prestressed or tilt-up concrete sandwich wall panels were examined. These components are used extensively in modern construction for cladding of framed building systems and often provide a significant level of protection from blast events.

To provide resistance against lateral loads, like those generated by wind or blast pressures, sandwich wall panels behave in a composite manner. Shear ties are utilized to achieve a level of composite action between the interior and exterior concrete layers. A variety of shear tie types are used in domestic construction. This study evaluated the performance of shear ties to understand the failure modes of sandwich wall panels conducted on the performance of shear ties. This report provides an in-depth examination of the shearing force - deformation characteristics of ties commonly used in the United States. This data can be used to develop predictive response models for sandwich panels under extreme events in future research.

The experimental results indicate that shear ties used in sandwich wall panels have a considerable variation strength, stiffness, and deformability. The maximum shear strength of the connectors average approximately 2,400 lbs with a minimum of 595 lbs and maximum of 6,008 lbs. The connectors varied in their responses from pseudo-rigid-brittle, elastic-brittle, elastic-plastic and plastic-hardening.

Tri-linear constitutive relationships were developed for each type of shear connector tested. The ranges of response were divided into three regions: elastic, plastic, and unloading. The elastic stiffness, K, was defined by the slope of the secant to 75% of the ultimate load, V_{max}. The yield displacement, Δ_y, was defined at the intercept of the ultimate load and the elastic curve. The ultimate displacement, Δ_u, was taken at the point when the strength decreases by 50% of the ultimate.

The constitutive relationships developed in this work are recommended for use to model the performance of shear tie-sandwich wall panel systems in a blast event. However, it is important to note that for traditional design, shear tie connectors are used to resist construction-related stresses during handling and placement as well as in-service live loads such as wind loads. Under these demands adequate overdesign is used to ensure that the ties remain in their elastic range. The data generated within this report are applicable in cases where sandwich wall systems are loaded above service conditions, i.e., in a blast event, and should be used accordingly. Consequently, the results generated herein are not recommended to predict sandwich wall system capability under conventional gravity and live load demands.

1 Introduction

1.1 Background

The information presented in this report represents the second phase of work under a Collaborative Research and Development Agreement (CRADA) between the Portland Cement Association (PCA) and the Air Force Research Laboratory (AFRL), Airbase Technology Division, Tyndall Air Force Base, Force Protection Branch (CRADA 05-119-ML-01). Sample donations have been provided from the Precast/Prestressed Concrete Institute (PCI) and Tilt-up Concrete Association (TCA) with support of their associated member companies. The overall research objective was to assess the inherent blast resistance of conventional concrete products. Conventional concrete products can be further defined as systems that are commercially and readily produced or engineered without any blast considerations/details in their design.

In the precast concrete wall industry, a large development thrust has been in green building technology and acquiring Leadership in Energy & Environmental Design (LEED) certification. With these requirements and guidelines the industry has turned to encased insulation to enhance the thermal performance of the building envelope. The insulation is sandwiched between an exterior and interior concrete layer to limit damage of the insulation and to ease construction. Shear ties are used to provide integrity between the interior and exterior concrete sections, referred to as wythes, as illustrated in Figure 1 and Figure 2. The shear ties allow the panels to be lifted and handled during building erection and allow the panels to behave as a composite against flexural demands. Varying the type and arrangement of the shear tie connectors allowed the panels to act as partially to fully composite.

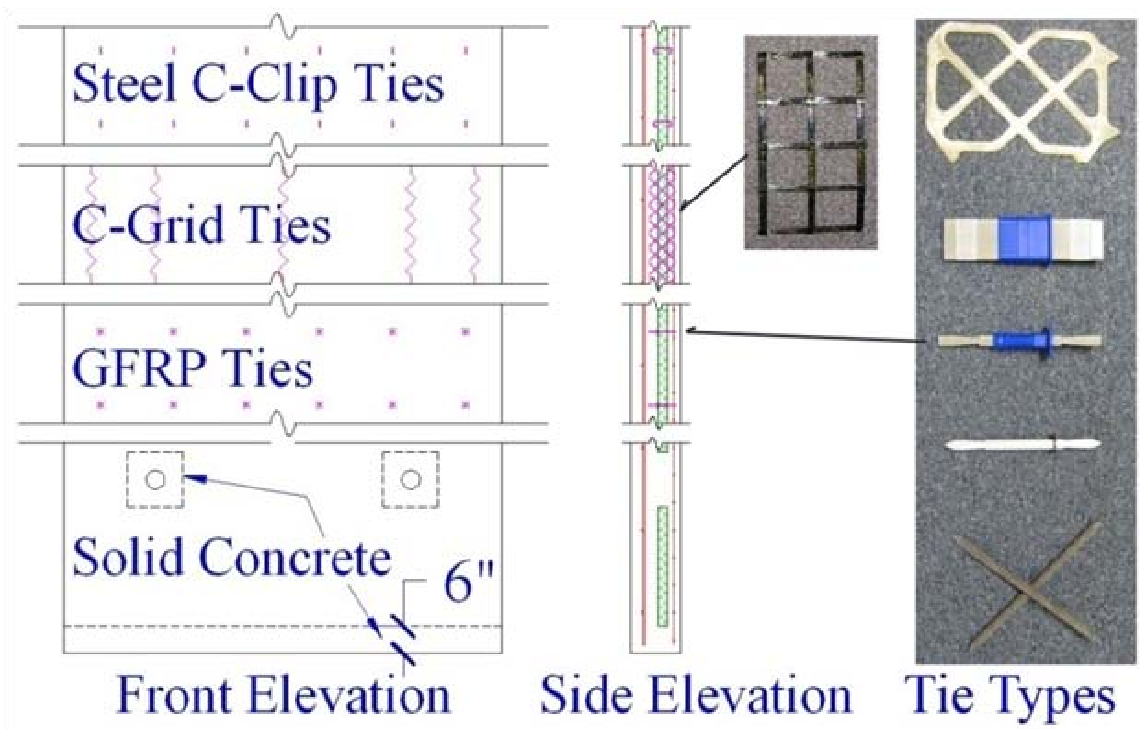


Figure 1. Shear Tie Connections in Sandwich Wall Panel

Figure 2. Shear Tie Installation in Foam in Sandwich Wall Panel

Shear ties are available in a variety of materials and configurations. These include carbon steel, stainless steel, galvanized carbon steel, carbon fiber reinforced polymers (CFRP), glass fiber reinforced polymer (GFRP), and basalt fiber reinforced polymer (BFRP). The various materials are chosen for their cost, thermal, or corrosion resistance benefits. Steel ties are commonly used when thermal and corrosion resistance is not a concern. These connectors are available at the lowest cost. When corrosion resistance is needed stainless steel or galvanized steel can be used at an increased cost. Unfortunately, steel has a high thermal conductivity which results in lower insulation properties for the walls. When high thermal requirements are specified and corrosion is a risk, GFRP, CFRP, or BFRP can be used.

Connectors are produced in a variety of configurations including trusses, pins, rods, and grids. From a force-based perspective, the shear capacity of the connectors can be determined either through first principles of engineering or through experimental validation. From a deformation-based perspective the variation in the shear connector configuration results in a range of deformation ability. For example, a FRP truss connector would likely produce a stiff, brittle response while a steel rod would likely result in a flexible response with large ductility. As a consequence the large displacement flexural response of a wall panel can vary significantly based on the type of connector used. To accurately predict the ultimate response of a sandwich panel subject to an increasing lateral pressure, the response of the shear tie connectors must be well defined. Due to the variety of connectors available, a consistent experimental approach was used to quantify and compare the effectiveness of shear ties in this research.

1.2 Research Objectives

A series of experiments were conducted on commercially available shear ties for use in precast concrete sandwich wall panels. A validated methodology is needed to predict the ultimate flexural strength of sandwich panels. To accomplish this, the characteristics of the ties used in the panel must be known. The objectives of the research were as follows:

- Quantify the shear strength of tie commercially-available domestic tie connectors for sandwich wall construction.

- Quantify the shearing force – deformation response of each connector type.

- Develop a simplified backbone response for each connector to facilitate modeling of composite action.

1.3 Scope

For traditional design, shear tie connectors are used to resist construction-related stresses during handling and placement as well as in-service live loads such as wind loads. Under these demands adequate overdesign is used to ensure that the ties remain in their elastic range. The data generated within this report are intended to be applicable in cases where sandwich wall systems are loaded above service conditions, i.e., in a blast event, and must be used accordingly. Consequently, the results generated herein are not recommended for use to predict sandwich wall system capability under conventional gravity and live load demands.

2 Shear Tie Mechanics

2.1 Shear Stress Demands

The flexural demands placed on sandwich panels produce internal compression, tension, and shear stresses. To support these internal demands as a composite section, the sandwich panel must have adequate tie reinforcement between the interior and exterior concrete wythes. This is accomplished by the placement of shear ties or the use of solid concrete zones between wythes. This study did not considered the solid concrete zone shear transfer mechanism, because the industry is moving away from these designs to ensure greater thermal properties in the wall panel. As illustrated in Figure 3, the flexural demands produce a shear demand perpendicular to the direction of loading. The magnitude of the shear can be computed using three techniques summarized as follows:

- Method 1 – Shear stress is computed from the maximum compression or tension force at the maximum moment region. This method is recommended by PCI [1997].

- Method 2 – Shear stress is computed from first principles. The derivation of this method is based on the elastic response of the member, thus the assumption is violated once cracking occurs.

- Method 3 – Shear stress is computed from the vertical shear acting on the panel. This method is recommended by American Concrete Institute (ACI) Committee 318 [2005].

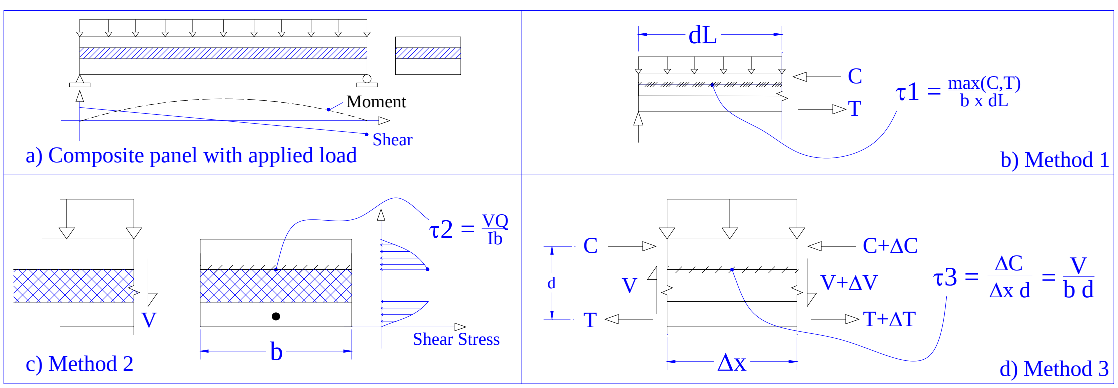


Figure 3. Shear Flow Free Body Diagrams

While Method 2 and 3 can be used; Method 1 is typically used for the the design of shear reinforcement for concrete sandwich wall panels following the practice of PCI [1997]. The maximum horizontal shear force is computed by taking the minimum of the compression and tension capacity of the section at midspan. The number of ties needed to resist the shear force must be placed on each half of the wall spanning from midspan to the support. To simplify the calculation the assumption is made that the entire exterior wythe is acting in compression.

The required shear capacity, $V_{required}$, can be computed as follows:

$$V_{required} = min\ (T, C), \text{ [kips]}$$

$$T = A_{ps} f_{ps} + A_s f_y$$

$$C = 0.85 f_c' b t_c$$

where

A_{ps} = Area of prestressing steel in tension wythe, [in^2]

A_s = Area of non-prestressed steel in tension wythe, [in^2]

f_{ps} = Stress in prestressing steel at ultimate flexural strength, [ksi]

f_y = Yield stress in non-prestressed steel [ksi]

f'_c = Concrete compressive strength, [ksi]

b = Width of wall panel, [in.]

t_c = Thickness of compression wythe, [in.]

To achieve a fully composite panel response the required number of shear ties, $N_{required}$, can then be computed using the following relationship:

$$N_{required} > V_{required} / V_{tie\text{-}capacity}$$

where

$V_{tie\text{-}capacity}$ = Strength of tie in shear [kips]

The strength of the shear ties commercially available in the U.S. are determined in this study and are summarized in section 2.4.

2.2 Approximate Shear Response of Ties

A simplified multi-linear curve was developed for each connector to model their response. The backbone curve was based on the average response computed for each connector type. The ranges of response are divided into three regions: elastic, plastic, and unloading. The elastic branch is defined by the secant to 75% of the ultimate load, V_{max}. The yield displacement, Δ_y, is defined at the intercept of the ultimate load and the elastic curve. The ultimate displacement, Δ_u, is taken at the point when the strength decreases by 50% of the ultimate. The elastic stiffness, K, is tabulated along with the displacement at the ultimate load, Δ_m. A schematic of the tri-linear curve development is illustrated in Figure 4. The measured properties from the experiments are summarized in Table 1 and Figure 5. These backbone curves can be used to model the shear response of connectors.

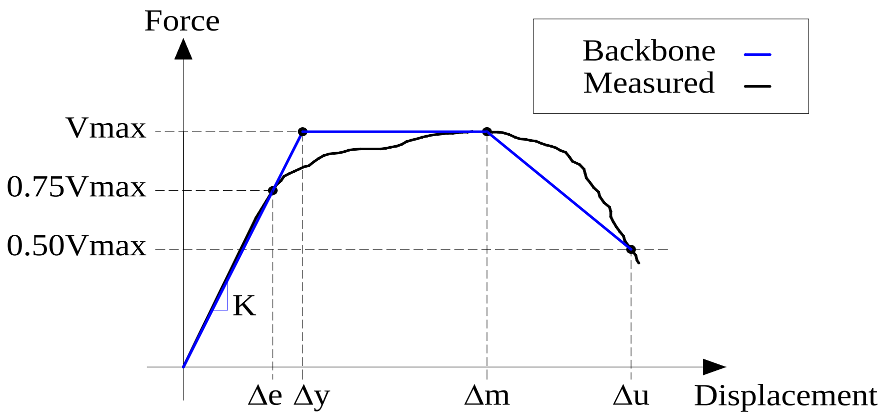


Figure 4. Backbone Development

Table 1. Backbone Parameters

Type	***K*** **[lb/in.]**	***Vmax**** **[lb]**	***Δe*** **[in.]**	***Δy*** **[in.]**	***Δm*** **[in.]**
A	95500	2164	0.017	0.023	0.076
B	22243	2675	0.090	0.120	0.326
C	1145	1104	0.723	0.964	1.034
D1	161806	3220	0.015	0.020	0.057
D2	1027099	2966	0.002	0.003	0.009
E	3058	769	0.189	0.251	0.646
F	2926	1737	0.445	0.594	0.529
G	1855	2064	0.834	0.676	0.984
H1	1794	803	0.336	0.448	0.637
H2	33719	1086	0.024	0.032	0.544
I	4304	3847	0.670	0.894	1.178
J	214584	4939	0.017	0.023	0.148
K	26830	2036	0.057	0.076	0.308
L	2320	1593	0.515	0.687	0.825

* Ultimate strength from average response curve
** Displacement at 50% of *Vmax* not measured

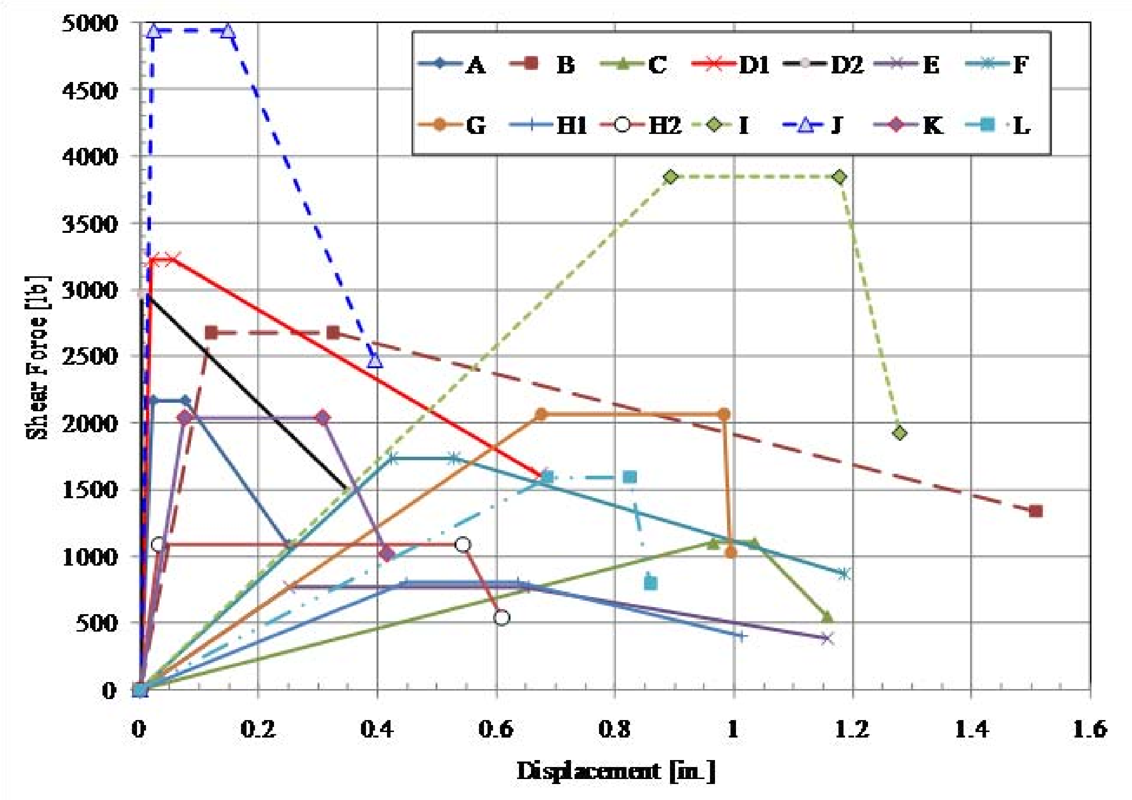


Figure 5. Multi-linear Approximations

2.3 Shear Performance Modeling

For a sandwich panel to behave as a true composite, the ties must have a rigid response under shear. Three of the connectors (D1, D2, and J) exhibited pseudo-rigid response up to an identifiable shear force. If the shear demand exceeds the limit of the rigid response, the ties deform, and the panels are expected to act independently beyond this point. This action is illustrated in Figure 6**.** As the tie becomes more flexible with progressive damage, the behavior of the panels approaches a non-composite response. The non-composite deformation response can be considered as a stacked plate where each plate is individually deforming with the same shape at the contact surface. Under this condition the shear ties would be subject to zero shear deformation at mid-span and with increasing shear deformation reaching a maximum at the supports. Knowing the relative slip at each connector, the shear demand in each shear tie can be determined from the constitutive relationship illustrated in Figure 5. Thus, the influence of tie stiffness and strength on ultimate panel capacity can be calculated. Further research on this topic is ongoing.

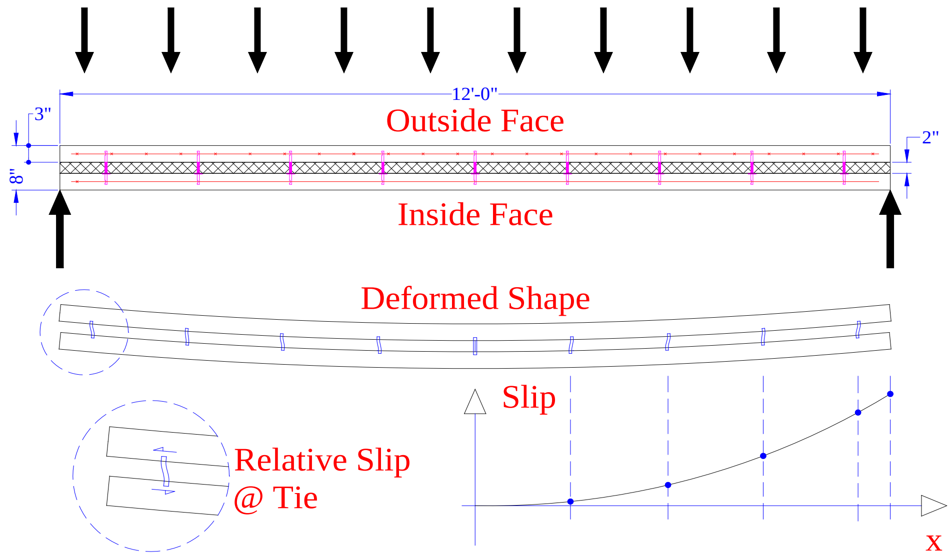


Figure 6. Shear Tie Deformation Demands

2.4 Shear Stiffness of Ties

In many cases the stiffness and failure mode of the shear tie can influence the ultimate flexural capacity of a sandwich panel. In a related study [Naito et. al., 2010] a series of sandwich wall panels were subjected to a monotonically increasing uniform load until failure occurred. The flexural response of the panel and the relative shear slip of the interior and exterior wythes were measured as illustrated in Figure 7. A number of different shear ties and reinforcement strategies were examined.

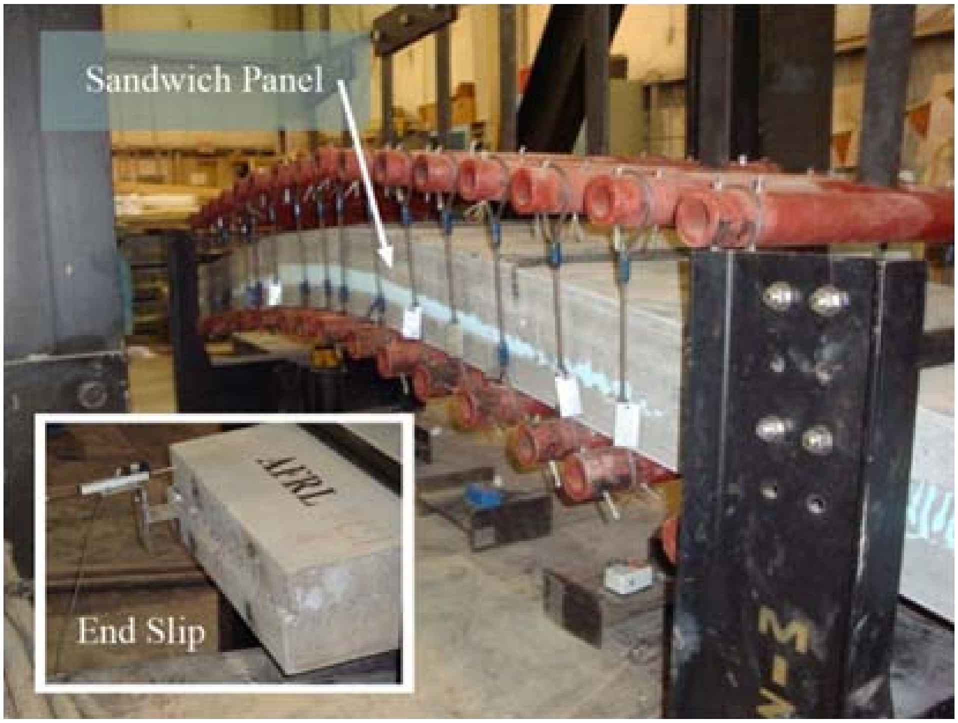
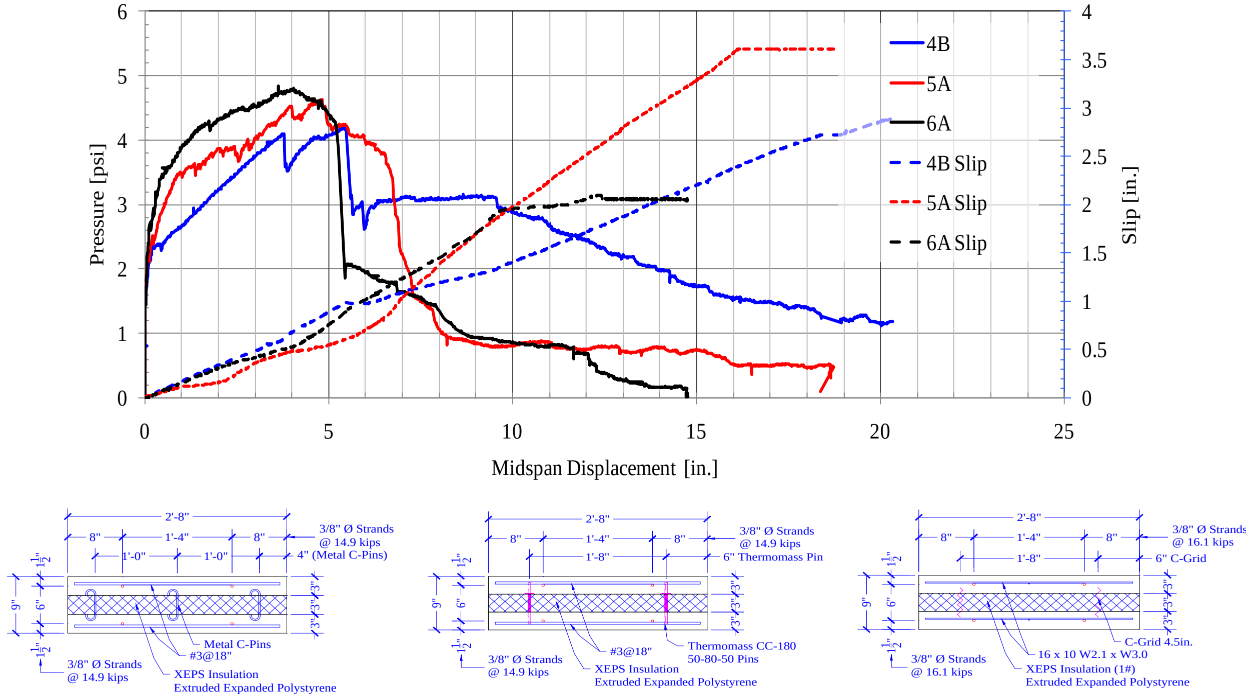


Figure 7. Typical Shear Slip of Exterior and Interior Wythes under Flexural Demands

The shear response of the ties influenced the shear failure modes of the panels. As an example the results of two experiments are presented in Figure 8. The results include the applied pressure and support rotations for three panels with the same flexural design. The tie type was varied between the panels. Panel PCS4 incorporated a flexible tie, panel PCS5 a moderately stiff tie, and PCS6 a stiff tie. The variation in tie types resulted in a change in the amount of relative slip measured between the wythes and a change in the ultimate capacity of the panels. As illustrated, the load-deformation behavior is sensitive to the tie type used. Between cracking and ultimate capacity, the use of a stiff tie increases the strength of the panel. To properly determine the load – deformation response, or resistance function, of a sandwich panel subjected to uniform loads, the connector shear load-deformation characteristics must be known.



PCS4

PCS5

PCS6

Figure 8. Flexural Panel Response and Relative Shear Slip

3 Experimental Program

3.1 Shear Tie Descriptions

Direct shear experiments were conducted on commercially-available connectors from the United States. The research program included both thermally-efficient polymer-based connections and traditional steel connections. The polymer connections included the following:

- (A) Glass Fiber Reinforced Polymer (GFRP) Delta Tie produced by Dayton Superior;
- (B) THERMOMASS® composite GFRP pins;
- (C) THERMOMASS® non-composite GFRP pins;
- (D) Altus Group CFRP Grid;
- (E) Universal Building products GFRP Teplo Tie; and
- (F) Universal Building products Basalt FRP RockBar.

The traditional steel connections included the following:

- (G) a galvanized steel C-clip by TSA,
- (H-1) galvanized C-clip produced by Dayton Superior,
- (H-2) stainless steel C-Clip produced by Dayton Superior,
- (I) galvanized steel M-Clip,
- (J) welded wire truss by Meadow Burke,
- (K) galvanized welded wire truss by Dayton, and
- (L) galvanized welded wire ladder by Dayton.

The overall test matrix is summarized in Table 2.

Table 2. Shear Tie Matrix

ID	Company	Tie Type	Material
A	Dayton	Delta Tie	GFRP Grid
B	THERMOMASS	Composite Tie	GFRP Pin
C	THERMOMASS	Non-Composite Tie	GFRP Pin
D-1[1]	Altus Group	C-Grid w/ EPS	CFRP Grid
D-2	Altus Group	C-Grid w/ XPS	CFRP Grid
E	Universal Building Products	TeploTie	GFRP Tie
F	Universal Building Products	RockBar	Basalt FRP Bar
G	TSA Manufacturing	C-Clip	Carbon Steel
H-1[2]	Dayton Superior	C-Clip	Galvanized Steel
H-2[3]	Dayton Superior	C-Clip	Stainless Steel
I	Dayton Superior	M-Clip	Galvanized Steel
J	Meadow Burke	Welded Wire Girder	1008 Steel
K	Dayton Superior	Single Wythe Truss	Hot Dipped Galvanized Steel
L	Dayton Superior	Single Wythe Ladur	Hot Dipped Galvanized Steel

[1] Two tests conducted.
[2] One test conducted.
[3] Four tests conducted.

The thirteen connectors are illustrated in Figure 9. The connectors as they were configured in each specimen are illustrated in Figure 10. The dimensions of each connector were measured and are reproduced in Figure 11.

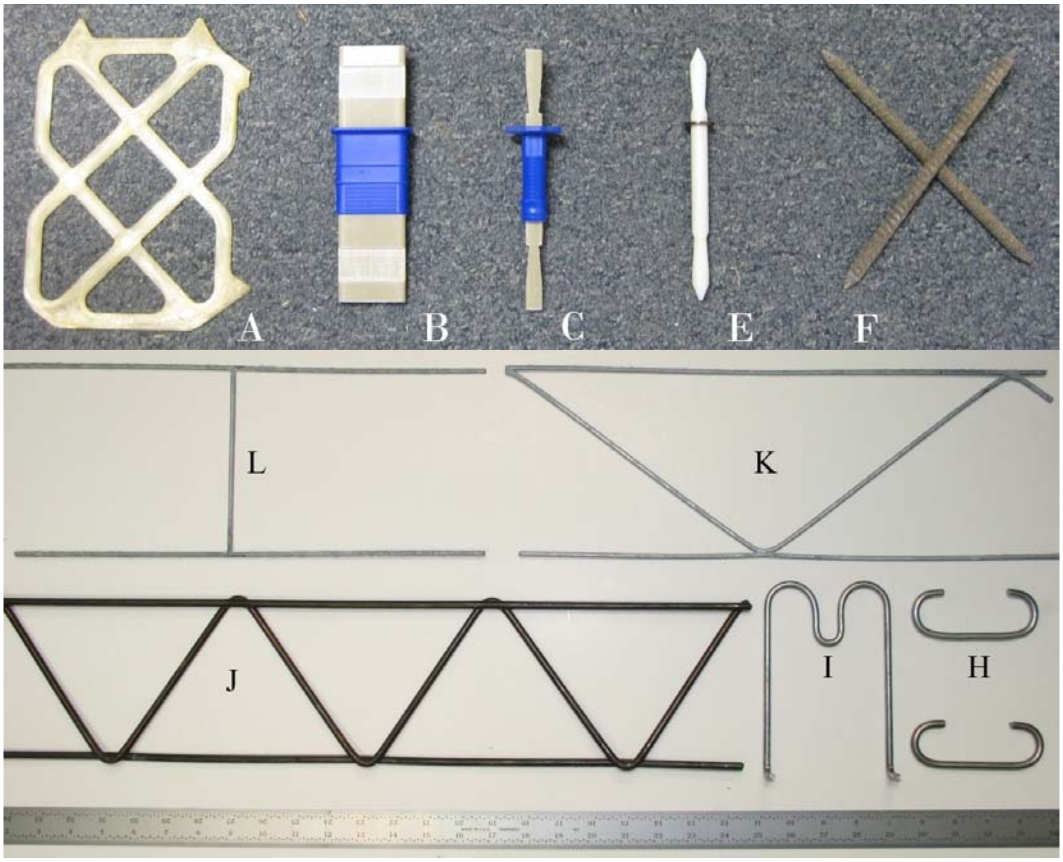


Figure 9. Photographs of Shear Tie Connectors

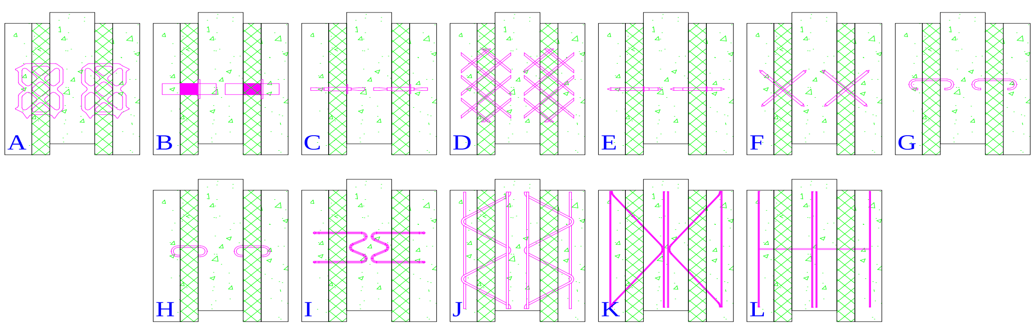


Figure 10. Tie Specimens (Drawn to Scale).

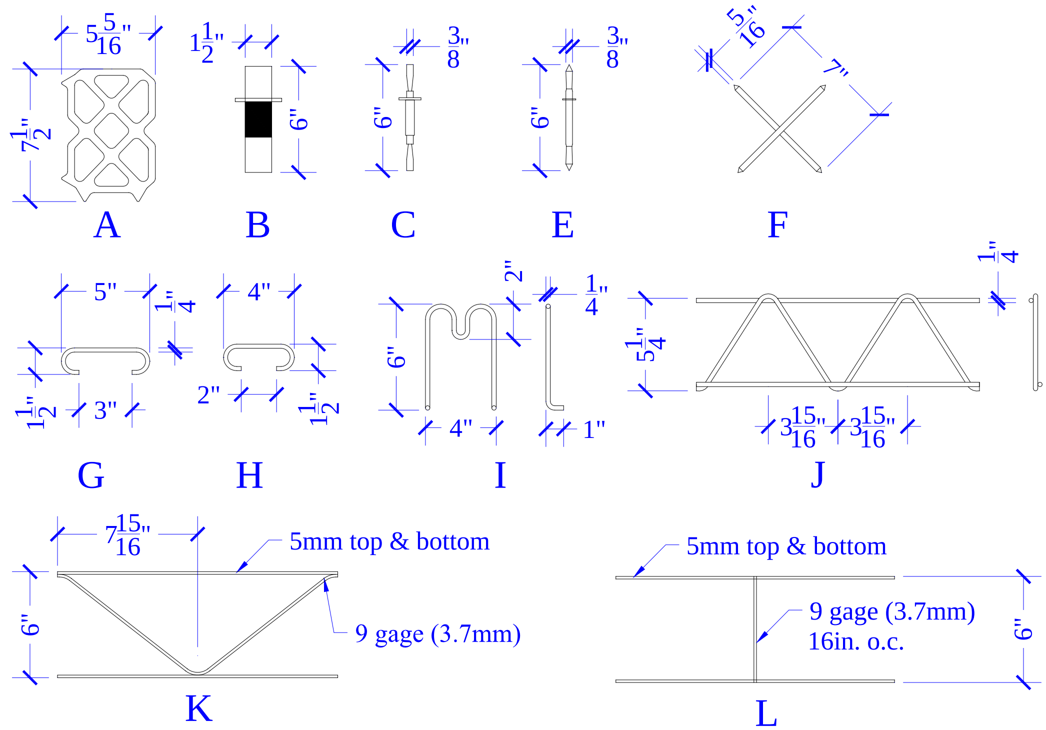


Figure 11. Measured Shear Tie Dimensions

3.2 Experiment Description

An experimental fixture was developed to evaluate the shear response of connectors. The loading fixture is illustrated in Figure 12. The specimen contains two connectors to minimize eccentricity and secondary demands on the connection during evaluation. An alternate fixture is specified in ASTM E488 Strength of Anchors in Concrete and Masonry Elements [2003]. The method illustrated in ASTM consists of a connector with a shear load applied directly to the connector. This creates an unrealistic boundary condition and produces prying forces on the connector resulting in a non-conservative measurement of strength. The fixture illustrated in Figure 12 was chosen to more accurately replicate the stresses acting on sandwich wall connectors under large flexural demands.

A minimum of three replicate tests were conducted for each connector type unless otherwise noted. Each tie was loaded to failure under a monotonically increasing displacement demand. This demand was used to replicate the conditions that would occur on ties located in a sandwich wall panel under a uniform blast-generated load. Blast pressure demands on walls are characterized by a high intensity dynamic load which exponentially decays over a short duration (typically, less than 100 msec). As a consequence the predominant flexural response of the panel occurs once during the initial positive pressure application. The shear ties are thus subject to the greatest demand once during the inbound cycle and subsequently are loaded to a lesser degree as the panel undergoes free vibration response. The cyclic response was not examined in this study.

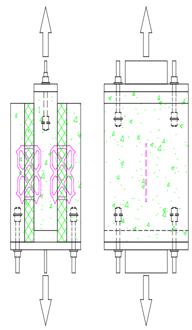


Figure 12. Testing Configuration

The experiments were conducted with a Material Testing System (MTS) closed-loop servo-controlled testing frame. The test frame was operated in displacement control of the MTS actuator. The specimens were examined at quasi-static loading rates. A displacement rate of 0.50 inch per minute was used for specimens A through F. Samples G through L were loaded at 0.25 inch per minute. The applied force was measured using a load cell in line with the MTS actuator. The shear strength tabulated in the report represents the force per connector (half of the load cell reading). The displacement was measured directly on the specimen using a linear variable displacement transducer (LVDT) as illustrated in Figure 12.

3.3 Test Specimens

All connectors were tested in a standardized specimen configuration. The tie specimens utilized 2 inches of insulation, which is commonly used in sandwich wall construction. Each specimen was fabricated from concrete with a compressive strength of approximately 4,000 psi at 28 days. The insulation consists of extruded polystyrene (XPS, also known as blue or pink board) in all cases except D1. Because of the low thermal conductivity of the C-grid system Expanded Polystyrene (EPS), also known as bead board, is commonly used for this connector. For completeness the C-grid connection was evaluated with both XPS and EPS insulation.

A standard embedment was used on each connector. To fit the connectors within the concrete specimen, 3-inch-thick exterior layers and a 5-inch-thick interior layer was used. The specimen details are illustrated in Figure 13.

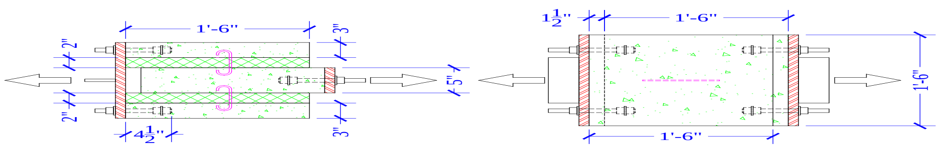


Figure 13. Shear Specimen Configuration

3.4 Concrete Strength

The specimens were fabricated in four sets. The first consisted of A, B, C, E, F, and G. The second group consisted of H, I, K, and L. The third group consisted of D connectors and the fourth group consisted of J and an addition H detail (H2-4). For the first group the compressive strength of the concrete used in fabrication of the specimens was measured prior to the shear test series and repeated after the series was completed. A concrete strength gain curve was developed in accordance with ACI 209 procedures and was used to estimate the strength of each specimen at time of testing. The strength gain formulation for the first group of tie tests is presented in Figure 14. The compressive strength for these specimen were based on the age of the concrete at the age of testing. The second group of tests was conducted over a period of two days. A compressive strength of 4,056 psi was measured at the time of shear testing in accordance with ASTM C 39 procedures. Group 3 and 4 were tested over a few days. Concrete compression strength tests were conducted in accordance with ASTM C39 following the last shear test. The compressive strength of group 3 was 10,357 psi. The compressive strength of group 4 was 5,081 ± 26 psi. The concrete compressive strength of each test is summarized along with the shear capacities are presented in Section 4.

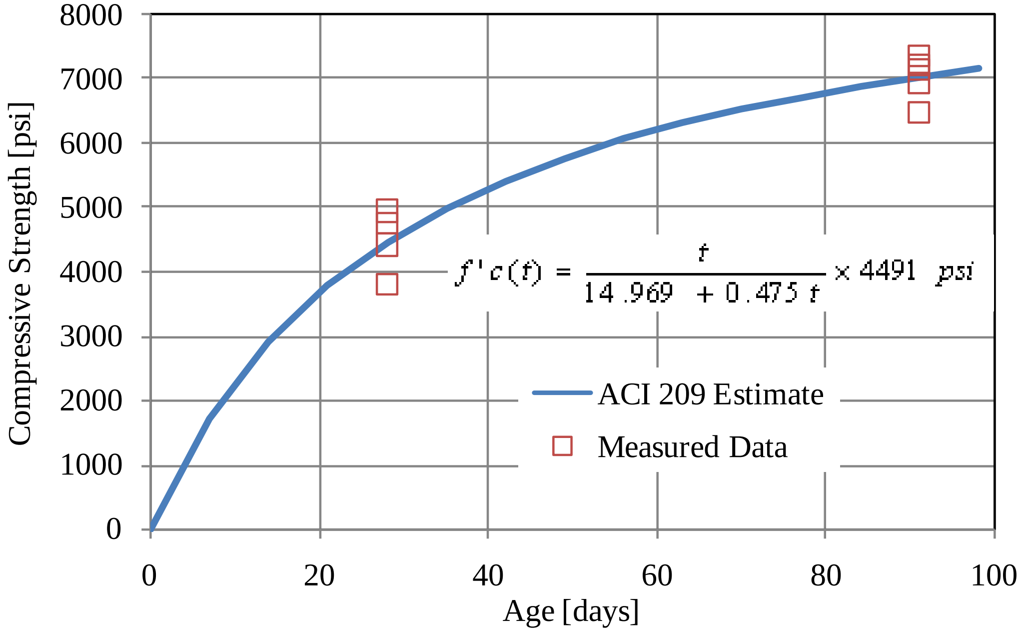


Figure 14. Estimate Concrete Strength Gain for Group 1 Specimens

4 Experimental Results

4.1 Summary of Results

A summary of the measured response of each experiment is presented in this section. The concrete compressive strength at the time of testing, f'_c, the peak shear strength, corresponding displacement, energy absorbed at the peak load, the average strength and the coefficient of variation on the strength are presented in Table 3. The shear strength measured represents the strength of one connector. The measured force was divided in half to account for the two tie experimental setup. The energy absorbed also represents the performance of one connector.

Table 3. Summary of Experimental Results

ID	Tie Type	f'_c [psi]	Peak Shear Strength [lbs]	Corresponding Displacement [in.]	Energy Absorbed at 0.2in. [lbs-in.]	Energy Absorbed at peak [lbs-in.]	Average Strength [lbs]
A1	GFRP Truss	6680	2632	0.070	340	107	2017
A2	GFRP Truss	6872	2424	0.115	399	220	
A3	GFRP Truss	7039	2672	0.014	340	30	
B1	GFRP Composite Pin	6680	2748	0.325	346	677	1905
B2	GFRP Composite Pin	6894	2634	0.340	417	774	
B3	GFRP Composite Pin	7039	2770	0.387	341	819	
C1	GFRP Non-Comp. Pin	6680	1119	1.030	104	748	1703
C2	GFRP Non-Comp. Pin	6894	1088	1.034	58	669	
C3	GFRP Non-Comp. Pin	7039	907	0.919	56	494	
D11	CFRP Truss (EPS)	10357	3362	0.054	534	150	3255
D21	CFRP Truss (EPS)	10357	3149	0.066	428	165	
D12	CFRP Truss (XPS)	10357	2871	0.007	443	18	2692
D22	CFRP Truss (XPS)	10357	3148	0.009	504	20	
D32	CFRP Truss (XPS)	10357	2058	0.433	195	656	
E1	GFRP Pin	6706	595	0.608	44	258	1924
E2	GFRP Pin	6894	825	0.650	76	408	
E3	GFRP Pin	7019	759	1.084	100	698	
F1	BFRP Bar	6706	2233	0.556	198	803	2523
F2	BFRP Bar	6894	1435	0.415	172	406	
F3	BFRP Bar	7039	1247	0.857	178	917	

Table 3. Summary of Experimental Results (Concluded)

ID	Tie Type	f'_c [psi]	Peak Shear Strength [lbs]	Corresponding Displacement [in.]	Energy Absorbed at 0.2in. [lbs-in.]	Energy Absorbed at peak [lbs-in.]	Average Strength [lbs]
G1	Galvanized C-Clip	6706	3831	1.519	99	2775	3407
G2	Galvanized C-Clip	6894	2452	0.661	228	1067	
G3	Galvanized C-Clip	7039	3938	1.408	156	3000	
H11	Galvanized C-Clip	4056	808	1.014	164	373	NA
H21	Stainless C-Clip	4056	944	0.942	114	434	1241
H22	Stainless C-Clip	4056	1085	0.665	135	441	
H23	Stainless C-Clip	4056	1356	0.673	193	456	
H24	Stainless C-Clip	5110	1579	0.629	164	616	NA
I1	M type	4056	4781	1.292	173	3503	4138
I2	M type	4056	3276	1.366	168	1320	
I3	M type	4056	4358	1.763	192	4236	
J1	Truss Girder	5110	4632	0.039	837	152	5278
J2	Truss Girder	5110	6008	0.066	918	213	
J3	Truss Girder	5110	5196	0.020	923	91	
K1	Wire Truss	4056	2047	0.472	300	587	2052
K2	Wire Truss	4056	2060	0.429	304	564	
K3	Wire Truss	4056	2048	0.447	328	539	
L1	Ladder Truss	4056	1580	1.155	60	937	1565
L2	Ladder Truss	4056	1808	0.868	131	940	
L3	Ladder Truss	4056	1307	0.710	87	537	

4.2 Discussion of Results

As illustrated in Table 3 and Figure 15, shear ties used in sandwich wall panels have a considerable variation strength, stiffness, and deformability. The maximum shear strength of the connectors average approximately 2,400 lbs with a minimum of 595 lbs and maximum of 6,008 lbs. The connectors exhibited pseudo-rigid-brittle, elastic-brittle, elastic-plastic, plastic-hardening and a variety of other responses.

The variation in shear force-deformation response was directly related to the variability in connector design. The FRP truss type connections (A and D) exhibited an elastic brittle response, because the shear behavior was dominated by FRP in tension. The steel wire truss

(K) exhibited an elastic plastic behavior, because the shear behavior was dominated by steel in tension. The steel M-clip (I) and the C-clip with adequate embedment (G) exhibited an elastic-plastic behavior at low shear deformations, because the leg of the connection is subjected to dowel action. As the deformation increased, the connector legs changed to a tension mode resulting in the observed increase in strength. A similar behavior was observed in the steel ladder connection (L); however, a smaller wire diameter caused the forces to be lower. Post-yield hardening did not occur in the standard C-clip details (H) because of the lack of embedment. Post-test inspection revealed that these connections failed due to pullout from the concrete. The FRP non-composite pins (C and E) exhibited an elastic-plastic response with minor hardening. These connections failed by combined flexure-tension demands at the concrete interface. The composite FRP pin (B) produced an elastic-plastic response with high deformation capacity. The failure mode of these connections was dominated by laminar fracture of the connector and a combined flexure-tension mode at the concrete interface.

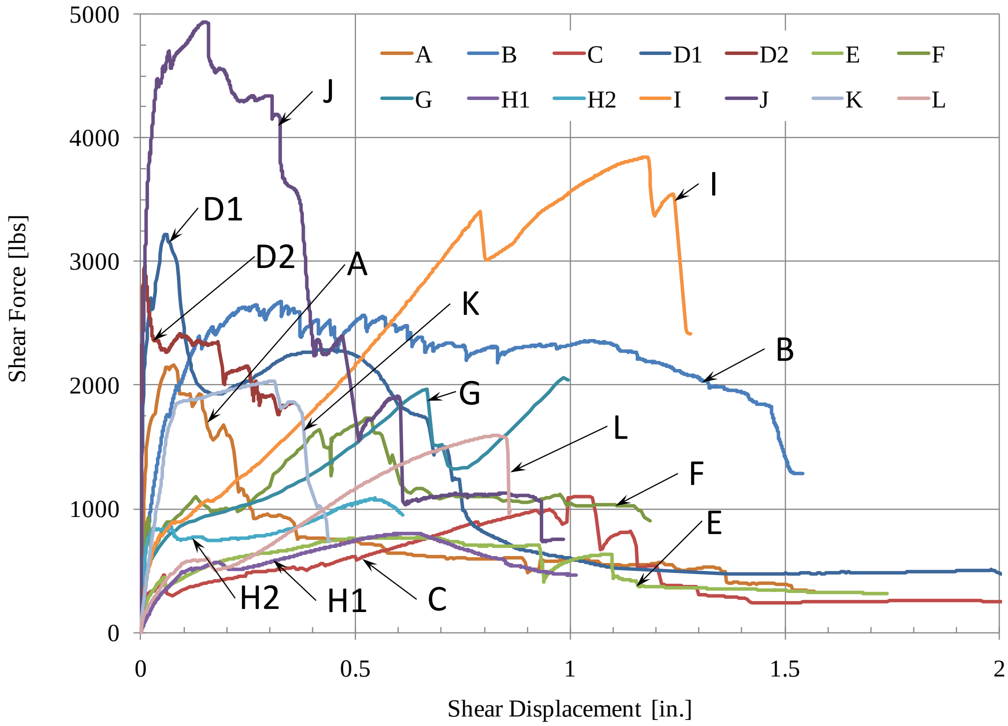


Figure 15. Summary of Shearing Force - Deformation Performance

It is important to note that for traditional design, shear tie connectors are used to resist construction-related stresses during handling and placement as well as in-service live loads such as wind loads. Under these demands adequate overdesign is used to ensure that the ties remain in their elastic range. The data generated within this report are applicable in cases where sandwich wall systems are loaded above service conditions, i.e., in a blast event, and should be used accordingly. Consequently, the results generated herein should not be used to predict sandwich wall system capability under conventional gravity and live load demands.

4.3 Summary Performance

The measured force displacement response curves for each shear tie type are presented in this section. As noted previously displacements were measured using an external LVDT, and force was measured using the MTS load cell. The response of each test is presented in graphical form. For each connector three test replicates are presented. An average of the three responses is computed for each data set. The average curve is included in each figure.

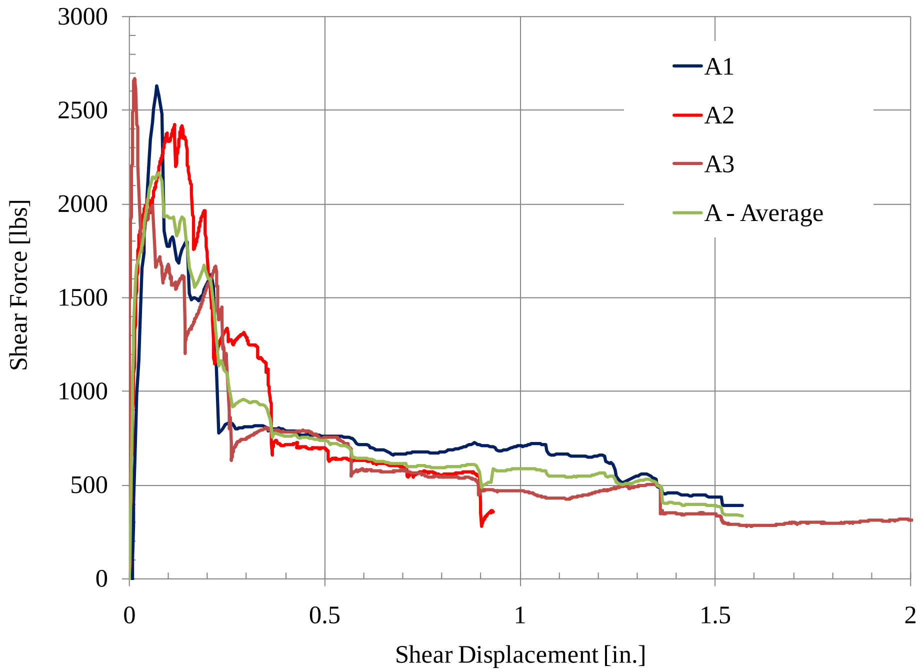


Figure 16. Tie A - Dayton Delta Tie Measured Shear-Deformation Response

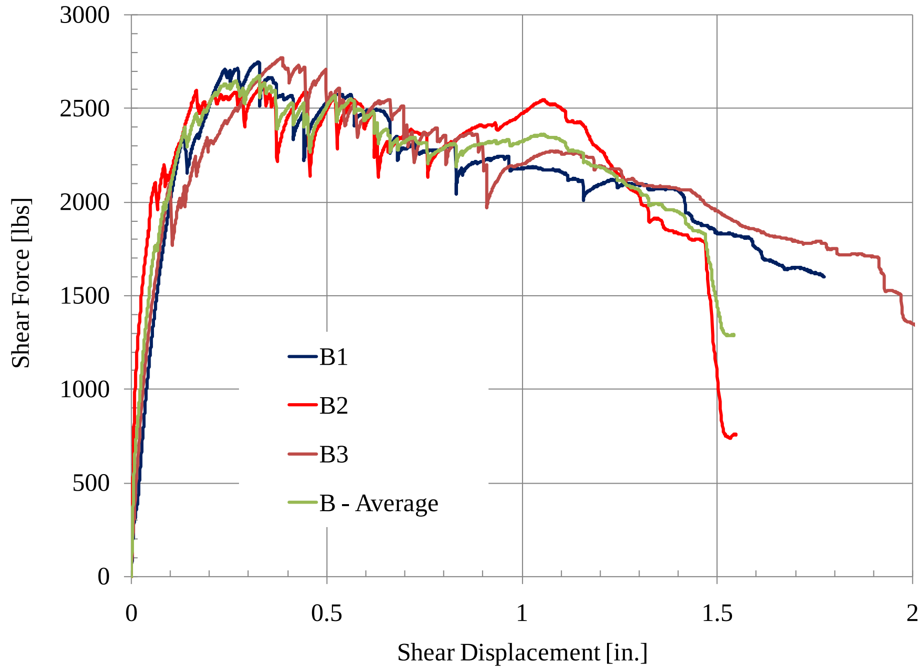


Figure 17. Tie B - THERMOMASS Composite Tie Measured Shear-Deformation Response

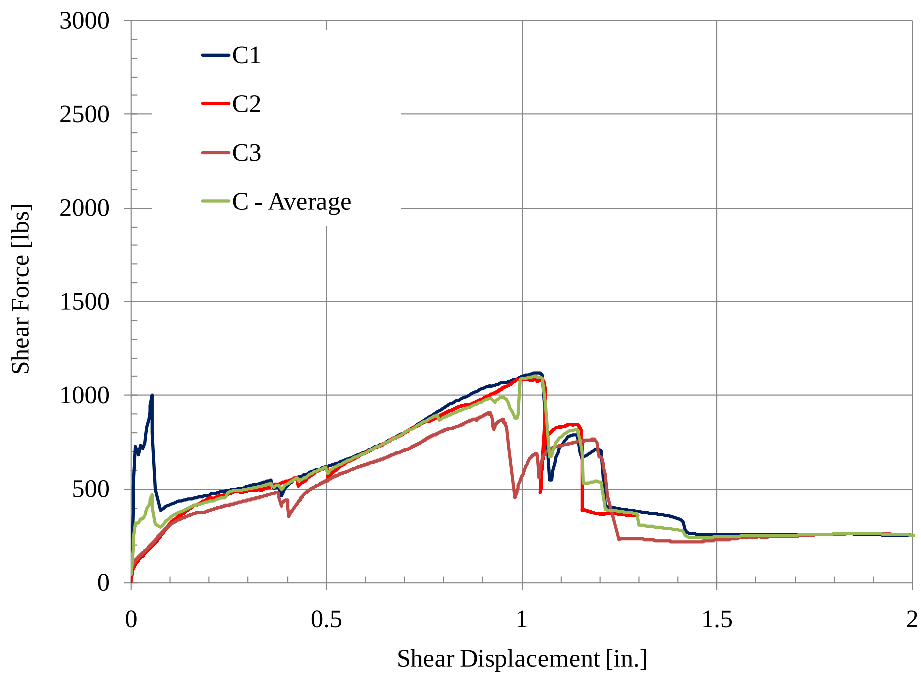


Figure 18. Tie C - THERMOMASS Non-Composite Tie Measured Shear-Deformation Response

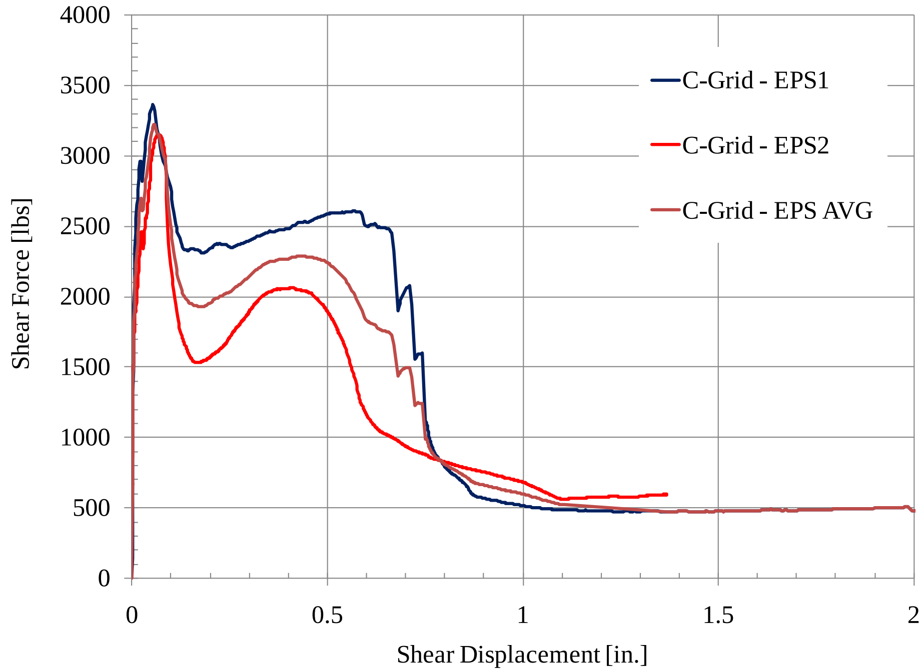


Figure 19. Tie D1 – Altus Group C-Grid (EPS) Measured Shear-Deformation Response

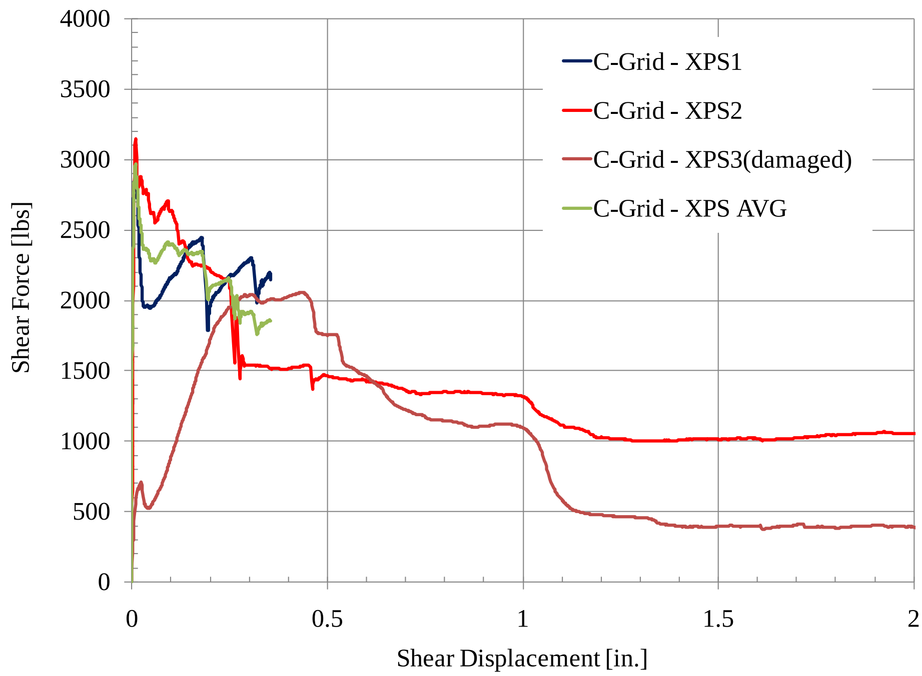


Figure 20. Tie D2 – Altus Group C-Grid (XPS) Measured Shear-Deformation Response

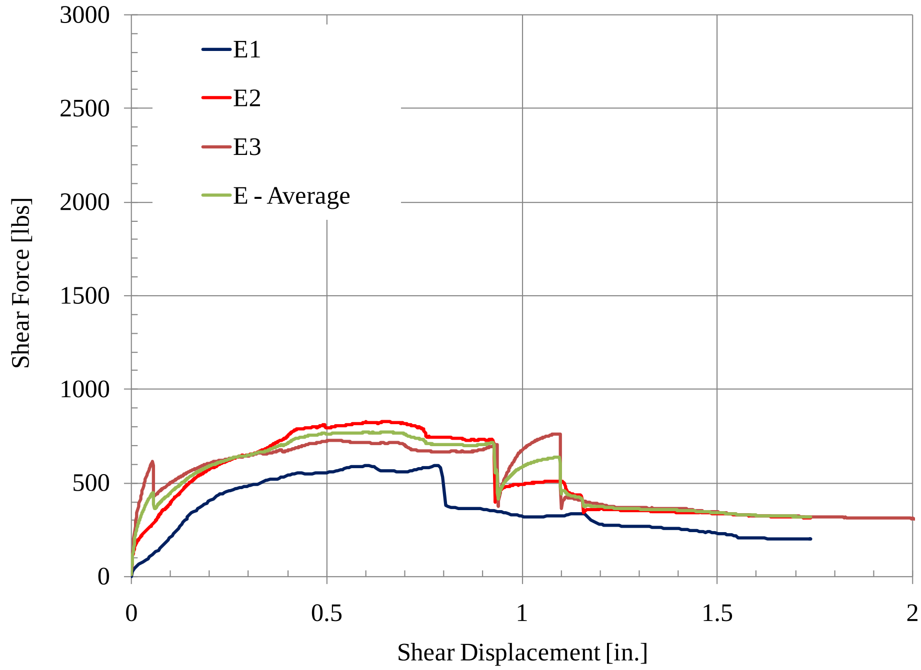


Figure 21. Tie E – Universal Building Products Teplo Tie Measured Shear-Deformation Response

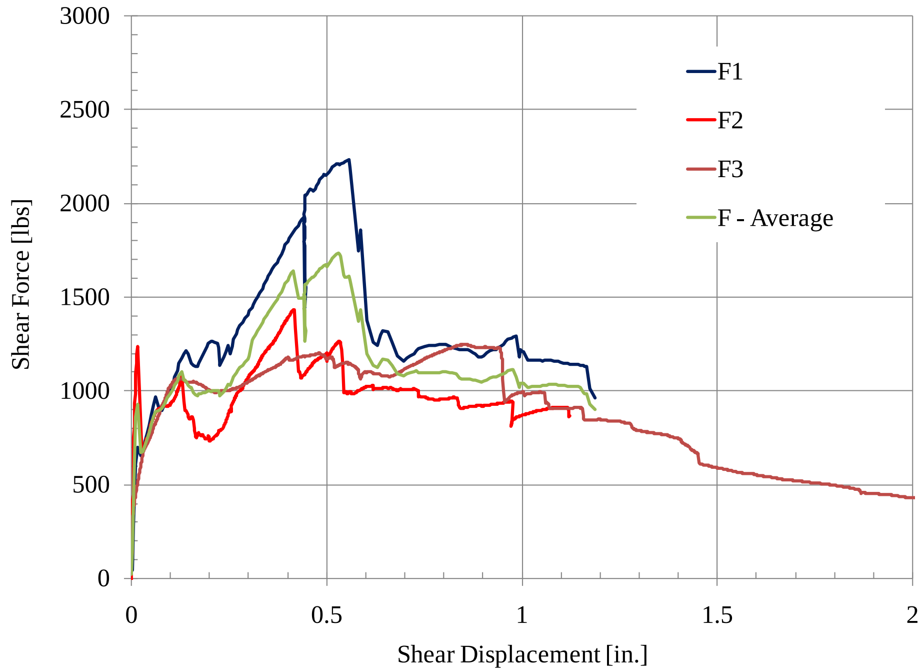


Figure 22. Tie F – Universal Building Products Rockbar Tie Measured Shear-Deformation Response

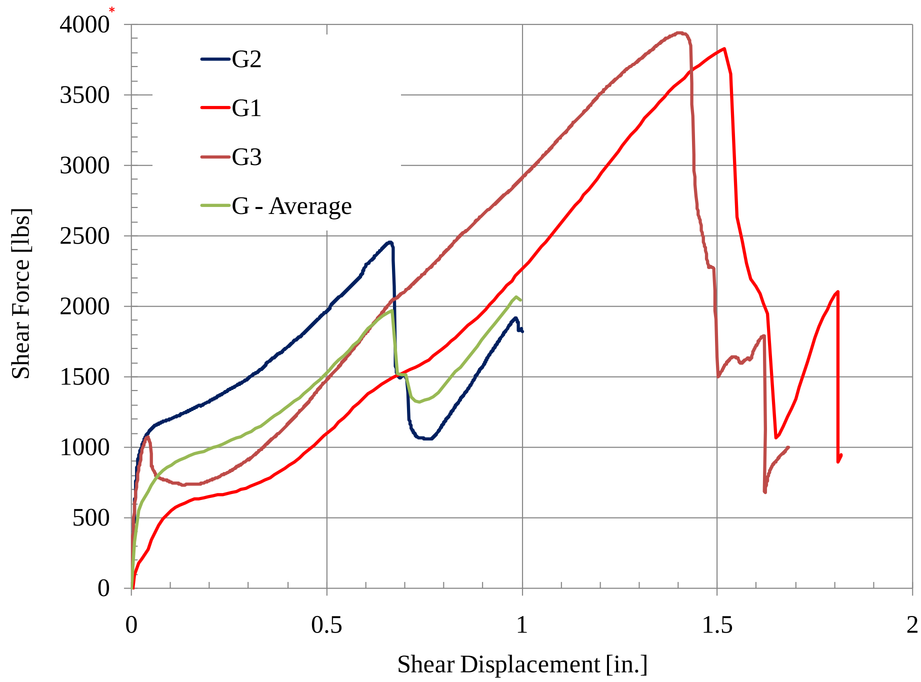


Figure 23. Tie G – Galvanized Steel C-Clip Tie Measured Shear-Deformation Response

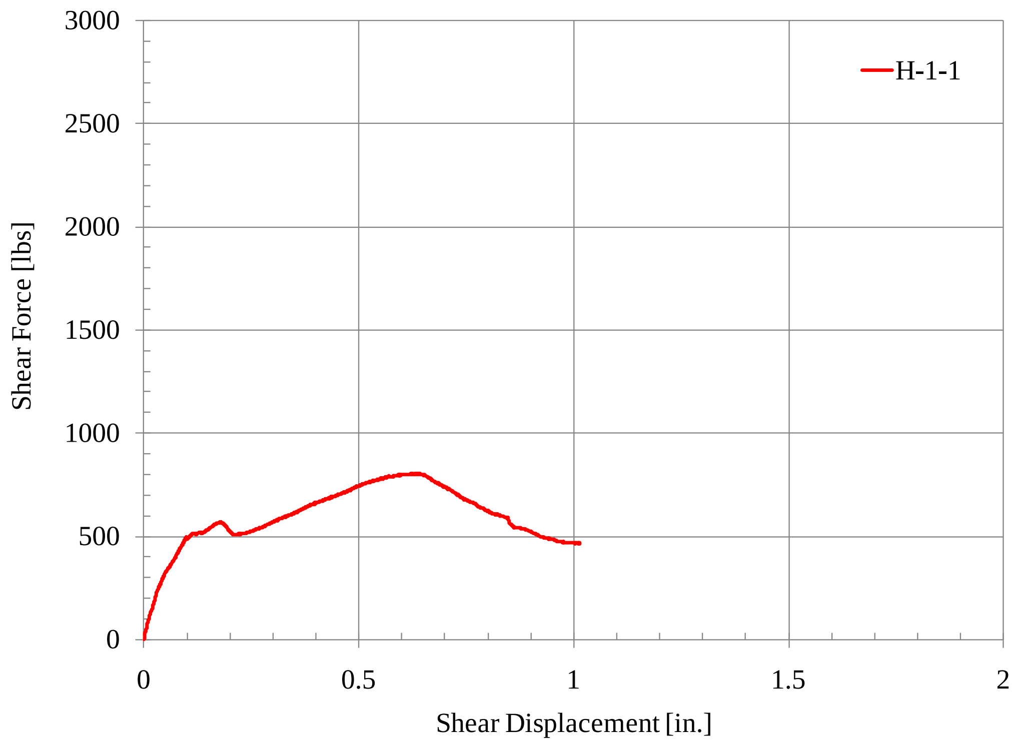


Figure 24. Tie H1 – Galvanized Steel C-Clip Tie Measured Shear-Deformation Response

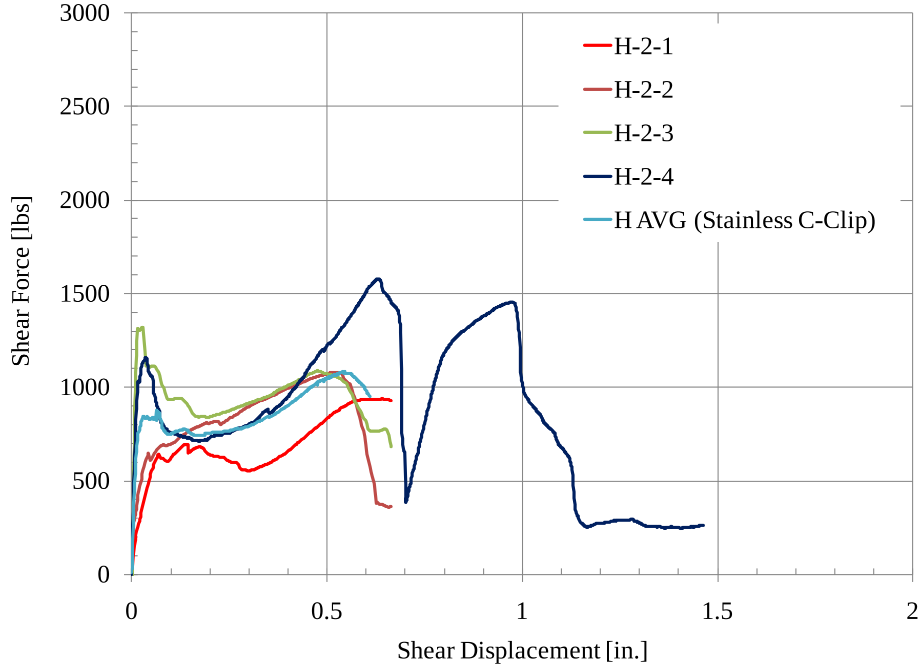


Figure 25. Tie H2 – Stainless Steel C-Clip Tie Shear-Deformation Response

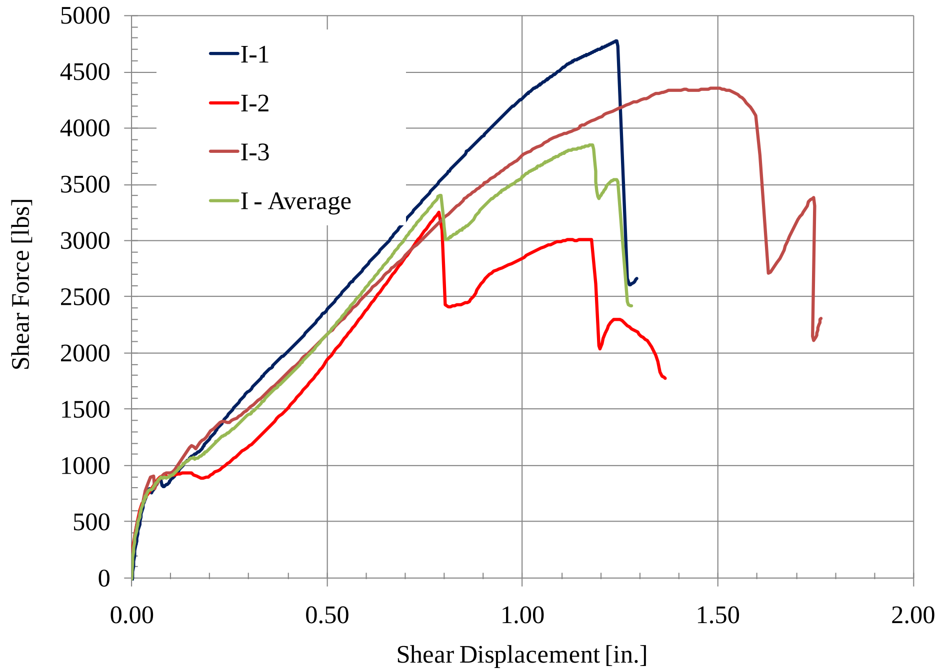


Figure 26. Tie I – M-Type Tie Shear-Deformation Response

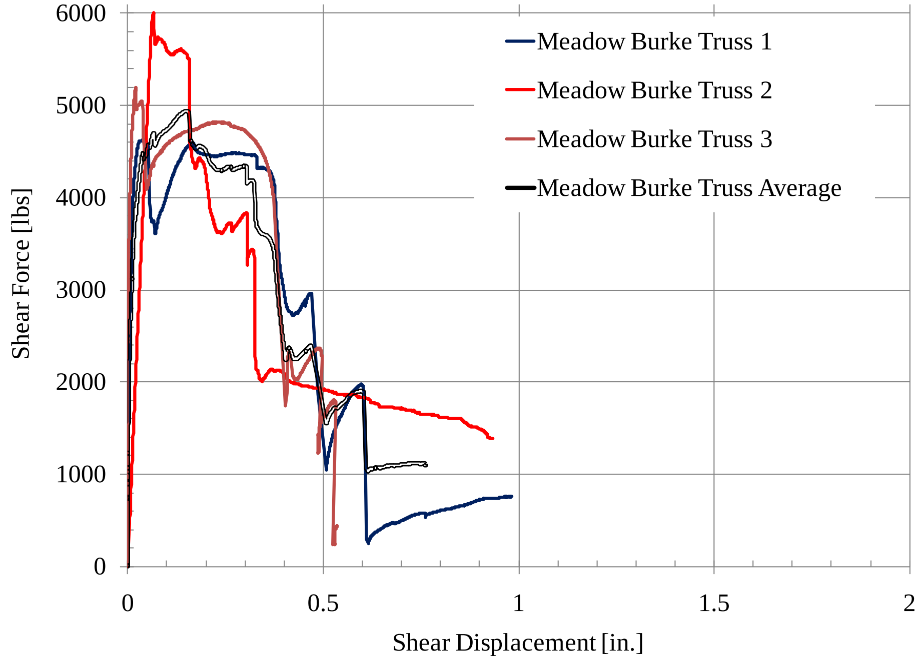


Figure 27. Tie J – Meadow Burke Truss Girder Shear-Deformation Response

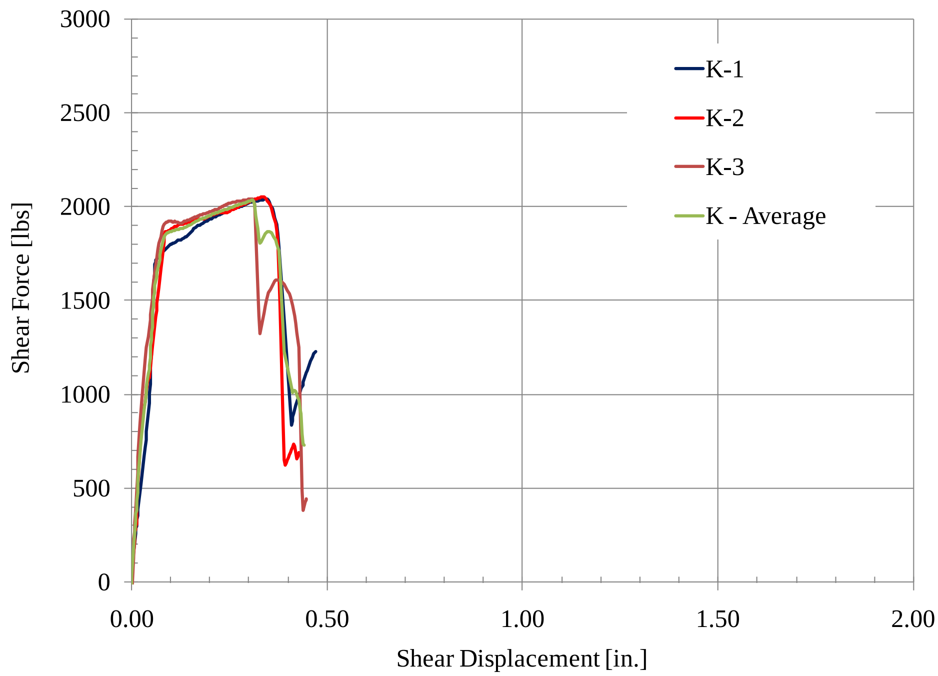


Figure 28. Tie K – Wire Truss Tie Shear-Deformation Response

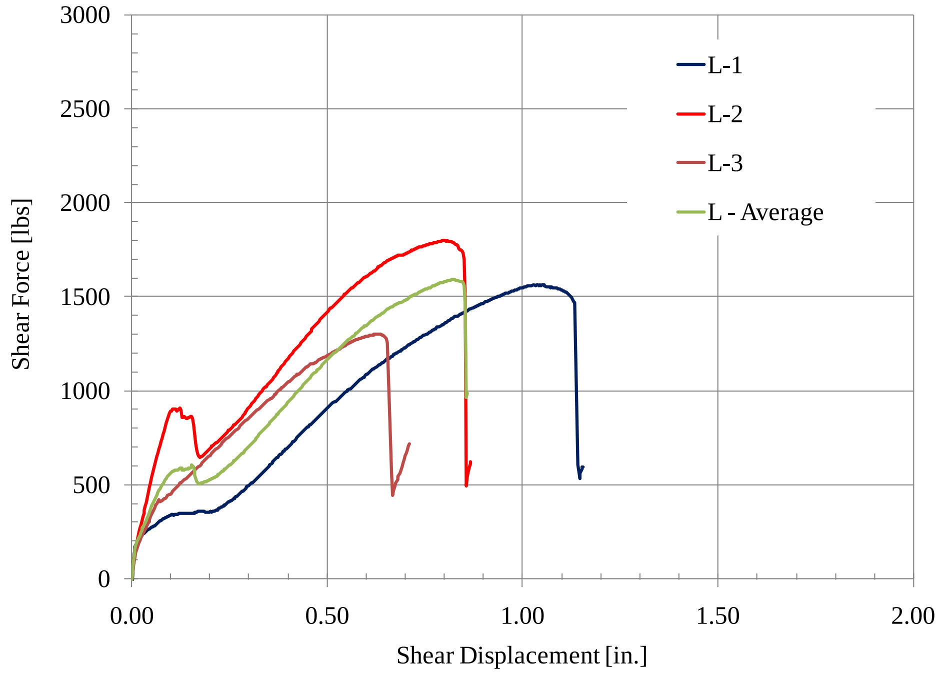


Figure 29. Tie L – Ladur Truss Tie Shear-Deformation Response

5 Conclusions and Recommendations

5.1 Conclusions

The responses of shear ties used in precast concrete sandwich wall panels were investigated to assess their inherent resistance at loads beyond service loads, a demand typical of a blast event. Domestically available shear tie systems were procured and tested in a shear fixture. The strength, stiffness and deformation capability of these ties were measured. The following conclusions were drawn from this study.

Shear ties used in sandwich wall panels vary considerably in strength, stiffness, and deformability. The maximum shear strength of the connectors tested in this study averaged approximately 2,400 lbs with a minimum of 595 lbs and max of 6,008 lbs.

Connector responses include pseudo-rigid-brittle, elastic-brittle, elastic-plastic and plastic-hardening. Tri-linear constitutive relationships were developed for each type of shear connector tested. The ranges of response were divided into three regions: (1) elastic, (2) plastic, and (3) unloading. The elastic stiffness, K, was defined by the slope the secant to 75% of the ultimate load, V_{max}. The yield displacement, Δ_y, was defined at the intercept of the ultimate load and the elastic curve. The ultimate displacement, Δ_u, was taken at the point when the strength decreases by 50% of the ultimate. These relationships can be used to.

5.2 Recommendations

The constitutive relationships developed in this work are recommended for use to model the performance of shear tie-sandwich wall panel systems in a blast event. However, it is important to note that for traditional design, shear tie connectors are used to resist construction-related stresses during handling and placement as well as in-service live loads such as wind loads. Under these demands adequate overdesign is used to ensure that the ties remain in their elastic range. The data generated within this report are applicable in cases where sandwich wall systems are loaded above service conditions, i.e., in a blast event, and should be used accordingly. Consequently, the results generated herein are not recommended to predict sandwich wall system capability under conventional gravity and live load demands.

Experimental validation of these relationships is recommended using full-scale blast experiments.

6 References

ACI Committee 209, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures," ACI 209-92, American Concrete Institute, 1997.

ACI Committee 318, "Building Code Requirements for Structural Concrete," ACI 318-05, 2005.

ASTM Standard C39, 2005, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," ASTM International, West Conshohocken, PA, 2005, DOI: 10.1520/C0039_C0039M-05E01, www.astm.org.

ASTM Standard E488, 2003, "Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements," ASTM International, West Conshohocken, PA, 2003, DOI: 10.1520/E0488-96R03, www.astm.org.

PCI Committee on Precast Sandwich Wall Panels, "State-of-the-Art Report of Precast/Prestressed Sandwich Wall Panels," PCI Journal, Vol.42, No.2, March-April, 1997.

Naito, C., Hoemann, J., Salim, H., Saucier, A., Shull, J., and Bewick, B., "Precast/Prestressed Concrete Experiments – Performance of Non-Load Bearing Sandwich Wall Panels," To be submitted to AFRL in Spring 2010.