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PERFORMANCE AND CHARACTERIZATION OF SHEAR TIES FOR USE IN INSULATED PRECAST CONCRETE SANDWICH WALL PANELS
PREPRINT

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PERFORMANCE AND CHARACTERIZATION OF SHEAR TIES FOR USE IN INSULATED PRECAST CONCRETE SANDWICH WALL PANELS

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ABSTRACT

Insulated precast concrete sandwich wall panels are commonly used for exterior cladding on building structures. The insulation is sandwiched between exterior and interior concrete layers to reduce the heating and cooling costs for the structure. The panels can be designed as composite, partially composite, or non-composite. Shear ties are used to achieve these varying degrees of composite action between the interior and exterior concrete layers. A variety of shear ties are available for domestic construction. An experimental study was conducted to assess the relative strength and response of these commercially available ties. Fourteen shear ties were examined, the failure modes and responses were quantified, and simplified engineer level multi-linear strength curves were developed for each connection. The results indicate that shear ties used in sandwich wall panels have considerable variation in strength, stiffness, and deformability. The maximum shear strength of the discrete ties averaged 10.5 kN (2357 lb) with a minimum of 5.52 kN (1241 lb) and maximum of 18.4 kN (4138 lb). The ties exhibited elastic–brittle, elastic–plastic, plastic–hardening, and a variety of other responses. The results were used to develop tri-linear constitutive relationships, which were used to approximate the flexural response of sandwich wall panels.

CE Database Subject Headings: Sandwich Panel, Prestressed Concrete, Composite, Test, Constitutive Model

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INTRODUCTION

In the precast concrete wall industry, a significant development thrust has been in "Green Building" and acquiring "LEED (Leadership in Energy and Environmental Design) certification." With these energy efficiency requirements and guidelines, the industry has increasingly turned to encased insulation to enhance the thermal performance of the building envelope while still maintaining construction speed. 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, or wythes, as illustrated in Figure 1. 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 ties controls the amount of composite action between the two wythes.

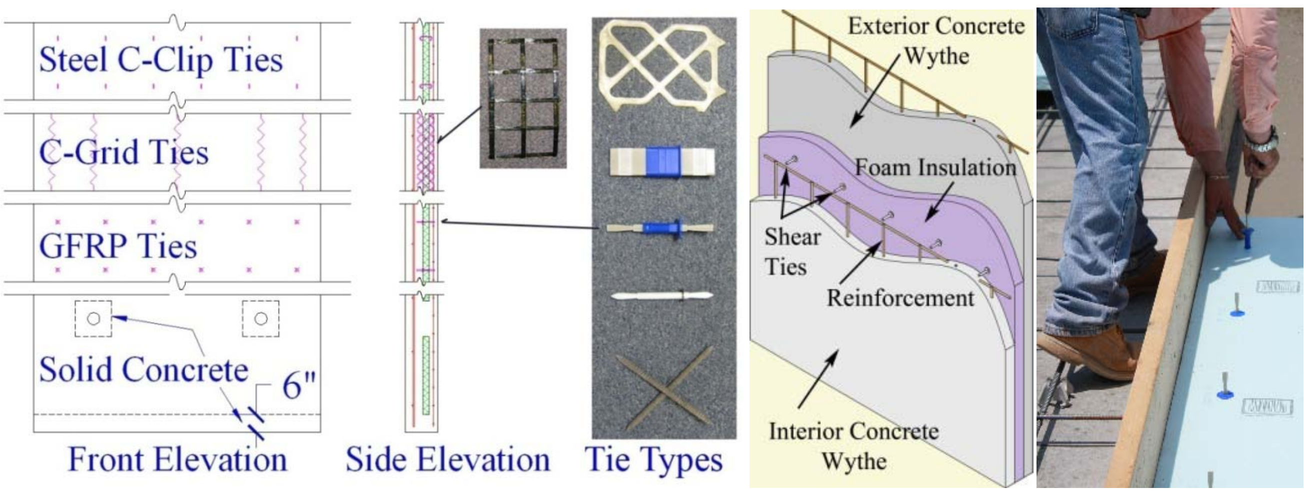


Figure 1: Shear ties in sandwich wall panels

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. Solid concrete zones provide a substantial means of achieving force transfer. However, commercial industries are moving away from these designs to reduce the bridging between wythes and increase overall thermal efficiency of the sandwich wall panel. As illustrated in Figure 2, maintaining the flexural demands requires the transfer of a shear force, through the ties, perpendicular to the direction of loading. The magnitude of the shear tie demand is commonly computed using one of three techniques. Method 1 computes the shear demand from the flexural capacity of the section. This method is recommended by the Precast/Prestressed Concrete Institute (PCI 1997). Method 2 computes the shear demand assuming elastic response and considering the first moment of the area above the shear tie. Since the derivation of this method is based on the elastic response of the member, the accuracy is poor after cracking. Method 3 approximates the shear tie demand from the transverse shear forces acting on the panel. This method is recommended by ACI 318 (2008).

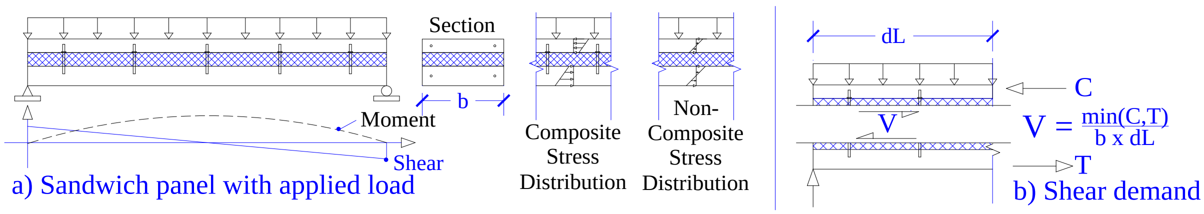


Figure 2: Shear flow force transfer

While Methods 2 and 3 can be used, in most cases the design of shear reinforcement for concrete sandwich wall panels follows the practice of PCI (1997). The maximum horizontal shear force is computed using the lesser of the compression or 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 depth of the exterior wythe is acting in compression.

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

$$V_{\text{required}} = minimum\ (T, C) \qquad \textit{Equation 1}$$

$$T = Tension = A_{ps} f_{ps} + A_s f_y \qquad \textit{Equation 2}$$

$$C == 0.85 f_c' b t_c \qquad \textit{Equation 3}$$

where A_{ps} is the area of prestressing steel in the tension wythe, A_s is the area of non-prestressed steel in the tension wythe, f_{ps} is the stress in the prestressing steel at ultimate flexural strength, f_y is the yield stress of the non-prestressed steel, f'_c is the concrete compressive strength, b is the width of the wall, and t_c is the thickness of the compression wythe.

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

$$N_{\text{required}} > \text{(Required Shear Capacity) / (Design Strength of a Single Shear Tie)} \qquad \textit{Equation 4}$$

For traditional panel design considerations, such as handling and wind, knowledge of the strength of the shear tie is adequate. In the extreme event, where panels are expected to reach their ultimate load capacity, both the strength and ductility capacity of the tie should be known. An example of an extreme event is an accidental or intentional explosion. This is a standard design condition for military facilities located near weapon storage depots, critical government or military facilities where anti-terrorism protection is a concern, or commercial facilities such as refineries or grain handling producers where gas or dust cloud explosions could occur. For these types of applications, there is potential for the panel to be loaded to and beyond its flexural capacity. To ensure safety to the occupants of the facility, the proper response of the shear ties within the panel must be considered. Furthermore, all three methods used to determine demand are based on the assumption that compatibility between the concrete wythes is maintained. If a flexible shear tie is used, the relative shear deformation could be very large at the required shear demand. Consequently, the design approach would no longer be valid. To accurately assess the response of the panel system under ultimate loads, the load deformation of the tie system used must be known. Commercially available shear ties were procured and experimentally evaluated. The results are used to develop simplified response curves, which are used for modeling the flexural response of wall panels.

SHEAR TIE SYSTEMS

Shear ties are available in a variety of materials and configurations. These include carbon steel, stainless steel, galvanized carbon steel, carbon-fiber-reinforced polymer (CFRP), glass-fiber-reinforced polymer (GFRP), and basalt-fiber-reinforced polymer (BFRP). The various materials are chosen for their cost and thermal or corrosion resistance benefits. Steel ties are commonly used when thermal and corrosion resistance is not a concern. These ties are available at the lowest cost. When corrosion resistance is needed, stainless steel or galvanized steel can be used at a premium. Steel, unfortunately, 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.

Shear ties are produced as trusses, pins, rods, and grids. The variation in shear tie configurations results in a broad range of deformation ability. For example, an FRP truss tie produces a stiff brittle response, whereas a thin steel rod results in a flexible response with large ductility. As a consequence, the flexural performance of a wall panel can vary significantly based on the tie used. To accurately predict the ultimate response of a sandwich panel subject to an increasing lateral pressure, the response of the shear ties must be well defined. The shear capacity of the ties can be determined either by analytical modeling of the mechanical and geometric properties of the tie or through experimental validation. Due to the variety of ties and their proprietary design, the flexibility and strength is not directly examined through modeling. Instead, a consistent experimental approach is used in this study to quantify and compare the effectiveness of shear ties.

EXPERIMENTAL PROGRAM

Direct shear experiments were conducted on ties commercially available in the United States. The research program includes both thermally efficient polymer-based connections and traditional steel connections. The polymer connections include: (A) 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. Traditional steel connections include (G) a galvanized steel C-clip, (H-1) galvanized steel C-clip, (H-2) stainless steel C-Clip, (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. Ties D, J, K, and L are distributed ties and are placed over the length of the panel. All other ties are designed to be discretely placed in the wall panel to achieve the desired capacity. The overall test matrix is summarized in Table 1. The dimensions of the fourteen ties are summarized in Figure 3.

Table 1: Shear Tie Matrix (Note: 1 in. = 25.4 mm)

ID	Company	Tie Type	Material	Size
A	Dayton	Delta Tie	GFRP Grid	Standard
B	THERMOMASS	Composite Tie	GFRP Pin	CC 150-50-50-50
C		Non-Composite Tie	GFRP Pin	MC 20/50
D-1[1]	Altus Group	C-Grid w/ EPS	CFRP Grid	C50 – 1.8 X 1.6
D-2		C-Grid w/ XPS	CFRP Grid	C50 – 1.8 X 1.6
E	Universal Building Products	TeploTie	GFRP Tie	10 mm dia. x 150 mm
F		RockBar	Basalt FRP Bar	7 in. x 5/16 in.
G	TSA Manufacturing	C-Clip	Galvanized Steel	5 in. x 1.5 in. wide
H-1[2]	Dayton Superior	C-Clip	Galvanized Steel	4 in. x 1.5 in.
H-2[3]		C-Clip	Stainless Steel	4 in. x 1.5 in.
I		M-Clip	Galvanized Steel	0.25 in. dia. – 6 in. tall
J	Meadow Burke	Welded Wire Girder	1008 Steel	0.25 in. dia. wire
K	Dayton Superior	Single Wythe Truss	Hot Dipped Galvanized Steel	See Figure 3
L		Single Wythe Ladur	Hot Dipped Galvanized Steel	See Figure 3

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

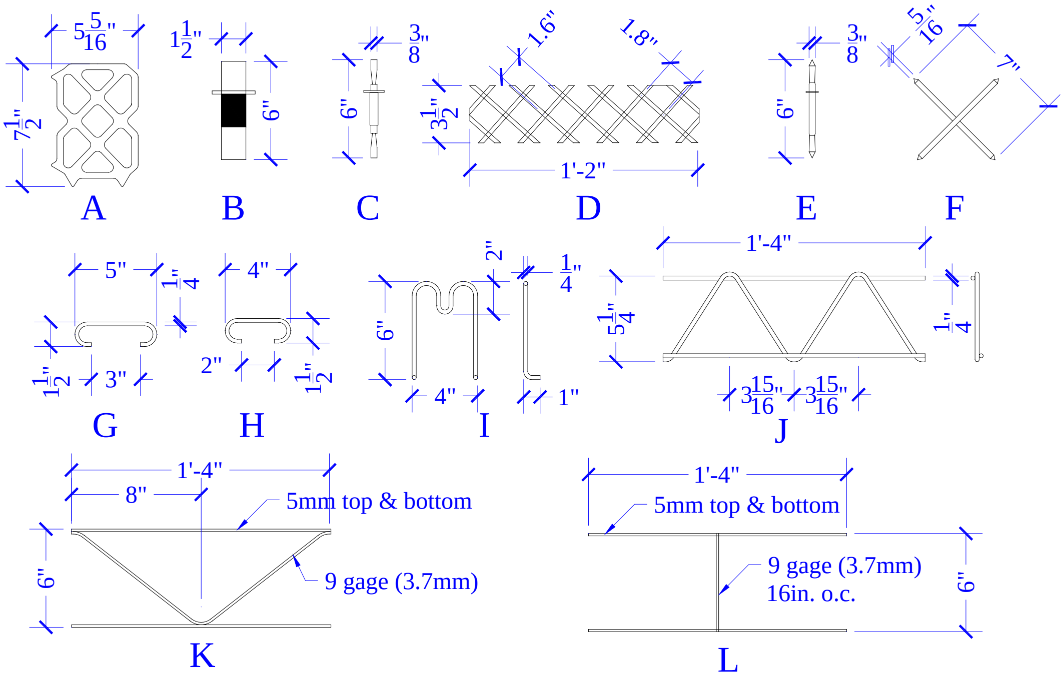


Figure 3: Measured shear tie dimensions (1 in. = 25.4 mm)

Experimental Setup

An experimental fixture was developed to evaluate the shear response of ties (Figure 4). The testing configuration contains two ties, each of the dimensions described in Table 1, to minimize eccentricity and secondary demands on the connection during evaluation. An alternate fixture specified in ASTM E488 Strength of Anchors in Concrete and Masonry Elements (2003) has been used for evaluation of ties. The method illustrated in ASTM E488 consists of a single tie with a shear load applied directly to the tie. This configuration produces prying forces directly on the shear tie that are not representative of the demands on shear ties in sandwich panels. The fixture illustrated in Figure 4 integrates the insulation foam and applies load to the tie through the concrete. This load transfer method more accurately replicates the demands acting on sandwich wall ties under large flexural demands and was used for the research program.

Three tests were conducted for each shear tie type unless noted. Each tie was loaded to failure under a monotonically increasing displacement demand. This demand is used to replicate the conditions that would occur on ties located in a sandwich wall panel under a uniform pressure load. Rate effects were not considered in this study. The relation between uniform pressure and blast-generated loads is further discussed in Biggs (1964). Blast pressure demands on walls are characterized by a high-intensity dynamic load that exponentially decays over a short duration (typically less than 100 msec). As a consequence, the predominant flexural response of the panel occurs during either the initial inbound or rebound response of the wall. Similarly, the shear ties are subject to the greatest demand during the initial inbound or rebound response and subsequently are loaded to a lesser degree as the panel undergoes free vibration response. Consequently, the cyclic response or elastic recovery was not examined in this study.

Figure 4: Shear tie testing configuration

The experiments were conducted with a universal test machine under displacement control. The specimens were examined at quasi-static loading rates. Higher rates of loading similar to those of blast were not conducted in this study. Higher rate loading would result in an increase in capacity due to the dynamic strength increase for the materials used. The quasi-static data presented can be used as a conservative estimate of tie strength. A displacement rate of 12.7 mm (0.50 in.) per minute was used in specimens A through F. Samples G through L were loaded at 6.4 mm (0.25 in.) per minute. Load was measured in line with the machine piston. The shear tie strengths in Table 2 and Table 3 are the force per shear tie (half of the load cell reading). The distributed ties are further divided based on the length tested. For these ties, the strength per shear tie length is presented. The displacement was measured directly on the specimen using a LVDT illustrated in Figure 4.

Test Specimen

All ties were tested in a standardized specimen configuration. The shear tie specimens use 50 mm (2 in.) of insulation, which is commonly used in sandwich wall construction. The insulation consists of extruded polystyrene (XPS, a.k.a. blue or pink board) in all cases but D1. Expanded Polystyrene (EPS, a.k.a. bead board) is commonly used for the C-grid shear tie (D1) to enhance the shear effectiveness of the panel assembly. For completeness, the C-grid connection is evaluated with both XPS and EPS insulation.

A standard embedment is used on each shear tie. To fit the ties within the concrete specimen, 7.6 cm (3 in.) exterior concrete layers and a 12.7-cm (5-in.) interior concrete layer were used. The specimen details are illustrated in Figure 5.

The specimens were cast from concrete with a specified minimum compressive strength of 27.6 MPa (4 ksi). The strength of each specimen was determined in accordance with ASTM C39 (2005). The samples were fabricated by TCA and PCI contractors who utilize site cast and plant cast concrete. The measured concrete strength varied from 27.6 MPa (4 ksi) to 68.9 MPa (10 ksi) and are summarized along with the shear capacities in Table 2 and Table 3.

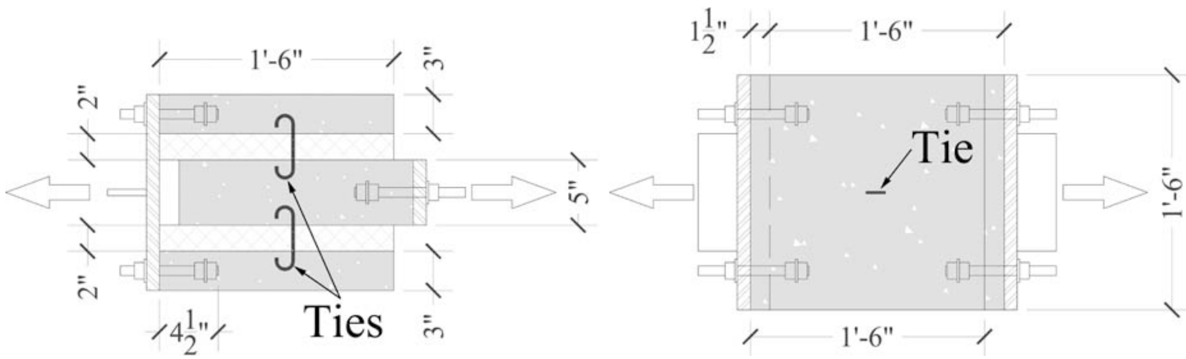


Figure 5: Shear specimen configuration (1 in. = 25.4 mm)

EXPERIMENTAL RESULTS

A summary of the measured responses 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 2 for discrete ties and Table 3 for distributed ties. The strength measured and energy absorbed represents the performance of one shear tie. The characteristic response of each shear tie was determined by averaging the force values at each displacement level for the group of connection results as illustrated in Figure 6. In general the results did not vary considerably within each group. Due to the averaging method used the peak force of the characteristic curves presented in Figure 7 are less than the peak force listed in Table 2 and Table 3 Further information on each test can be found in Naito, et al. (2009).

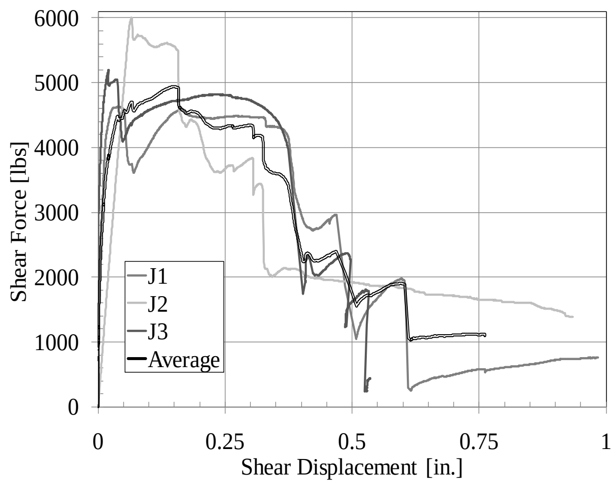


Figure 6: Typical determination of average response (1 in. = 25.4mm)

Table 2: Summary results discrete ties (Note: 1 in. = 25.4 mm, 1 lb = 4.45 N)

ID	Tie Type	f'_c (psi)	Per Shear Tie					
			Peak Shear Strength (lb)	Corresponding Displacement (in.)	Energy Absorbed at 0.2 in. (lb-in.)	Energy Absorbed at peak (lb-in.)	Average Shear Strength (lb)	COV
A1		6680	2632	0.070	340	107		
A2	GFRP Truss	6872	2424	0.115	399	220	2017	6.6%
A3		7039	2672	0.014	340	30		
B1		6680	2748	0.325	346	677		
B2	GFRP Composite Pin	6894	2634	0.340	417	774	1905	3.8%
B3		7039	2770	0.387	341	819		
C1		6680	1119	1.030	104	748		
C2	GFRP Non-Comp. Pin	6894	1088	1.034	58	669	1703	6.7%
C3		7039	907	0.919	56	494		
E1		6706	595	0.608	44	258		
E2	GFRP Pin	6894	825	0.650	76	408	1924	6.2%
E3		7019	759	1.084	100	698		
F1		6706	2233	0.556	198	803		
F2	BFRP Bar	6894	1435	0.415	172	406	2523	20.8%
F3		7039	1247	0.857	178	917		
G1		6706	3831	1.519	99	2775		
G2	Galvanized C-Clip	6894	2452	0.661	228	1067	3407	24.3%
G3		7039	3938	1.408	156	3000		
H11	Galvanized C-Clip	4056	808	1.014	164	373	NA	NA
H21		4056	944	0.942	114	434		
H22	Stainless C-Clip	4056	1085	0.665	135	441	1241	22.8%
H23		4056	1356	0.673	193	456		
H24		5110	1579	0.629	164	616	NA	NA

Table 2: Summary results discrete ties (Note: 1 in. = 25.4 mm, 1 lb = 4.45 N)

ID	Tie Type	f'_c (psi)	Per Shear Tie					
			Peak Shear Strength (lb)	Corresponding Displacement (in.)	Energy Absorbed at 0.2 in. (lb-in.)	Energy Absorbed at peak (lb-in.)	Average Shear Strength (lb)	COV
I1		4056	4781	1.292	173	3503		
I2	M type	4056	3276	1.366	168	1320	4138	18.8%
I3		4056	4358	1.763	192	4236		

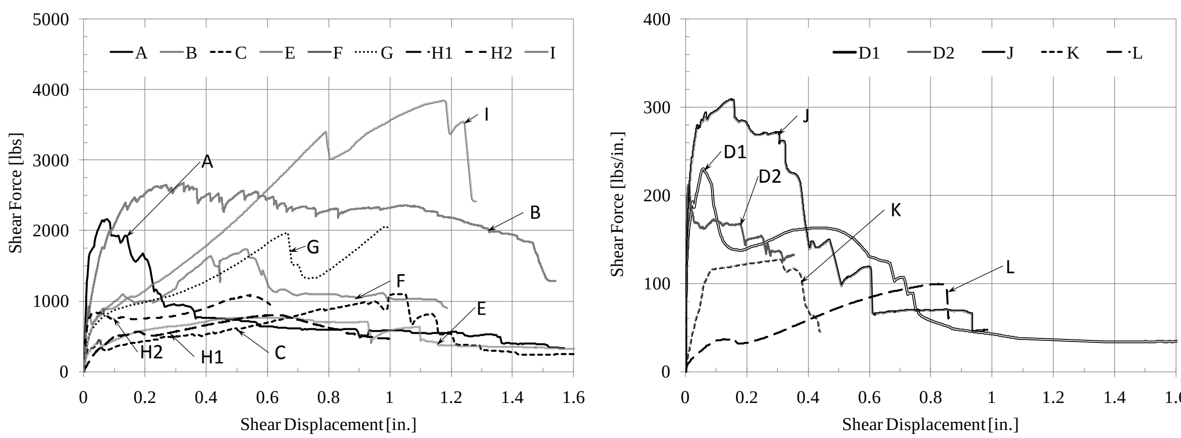
Table 3: Summary results distributed ties (Note: 1 in. = 25.4 mm, 1 lb = 4.45N)

ID	Tie Type	f'_c (psi)	Per Unit Length of Shear Tie					
			Peak Shear Strength (lb/in.)	Corresponding Displacement (in.)	Energy Absorbed at 0.2 in. ((lb/in.)-in.)	Energy Absorbed at peak ((lb/in.)-in.)	Average Shear Strength (lb/in.)	COV
D11	CFRP Truss (EPS)	10357	240	0.054	38	11	233	NA
D21		10357	225	0.066	31	12		
D12		10357	205	0.007	32	1		
D22	CFRP Truss (XPS)	10357	225	0.009	36	1	192	21.1%
D32		10357	147	0.433	14	47		
J1		5110	289	0.039	52	10		
J2	Truss Girder	5110	375	0.066	57	13	330	13.1%
J3		5110	325	0.020	58	6		
K1		4056	128	0.472	19	37		
K2	Wire Truss	4056	129	0.429	19	35	128	0.4%
K3		4056	128	0.447	21	34		
L1		4056	99	1.155	4	59		
L2	Ladder Truss	4056	113	0.868	8	59	98	16.0%
L3		4056	82	0.710	5	34		

Discussion of Results

As described in Table 2 and illustrated in Figure 7, shear ties used in sandwich wall panels have a considerable variation in strength, stiffness, and deformability. The average shear strength of the discrete ties is 10.5 kN (2,360 lb) with a minimum average of 5.5 kN (1,240 lb) and maximum average of 18.4 kN (4,138 lb). The maximum average shear response of the distributed ties is 34.0 kN/m (194 lb/in.) with a minimum average of 17.2 kN/m (98 lb/in.) and maximum of 57.8 kN/m (330 lb/in.). The ties exhibited elastic–brittle, elastic–plastic, plastic–hardening, and a variety of other responses.

The variation in shear–deformation response is directly related to the variability in shear tie design. The FRP truss type connections (A and D) exhibited an elastic–brittle response because the shear behavior is dominated by FRP in tension. The steel wire truss (K) exhibited an elastic–plastic behavior because the shear behavior is 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 as the leg of the connection is subject to dowel action. As the deformation increased, the shear tie 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, the forces are lower due to a smaller wire diameter. Post-yield hardening did not occur in the standard C-clip details (H) due to 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 moderate deformation capacity. The failure mode of these connections was dominated by laminar fracture of the shear tie and a combined flexure–tension mode at the concrete interface. Use of EPS over XPS (compare D1 to D2) was shown to increase the shear strength of the shear tie. The shear strength was influenced by the foam type used. This occurred due to the greater roughness provided by EPS over that of XPS.



a) Discrete ties b) Distributed ties

Figure 7: Shear-deformation performance (1 in. = 25.4 mm, 1 lb = 4.45 N)

Approximate Shear Response of Ties

To model the response of the ties, a simplified multi-linear curve was developed for each shear tie. The backbone curve was based on the average response computed for each shear tie type. The ranges of response are divided into three regions: (1) elastic, (2) plastic, and (3) unloading. The elastic branch is defined by the secant to 75% of the ultimate load, Vmax. The yield displacement, Δy, is defined at the intercept of the ultimate load and the elastic curve. The ultimate displacement, Δu, 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, Δm. A schematic of the tri-linear curve development is illustrated in Figure 8. The measured properties from the tie tests are summarized in Figure 9 and Table 4. These backbone curves can be used to model the shear response of ties.

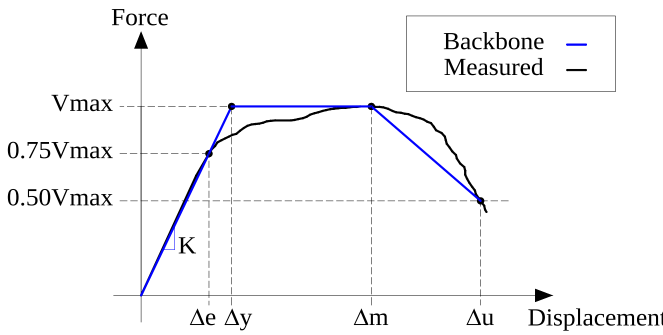
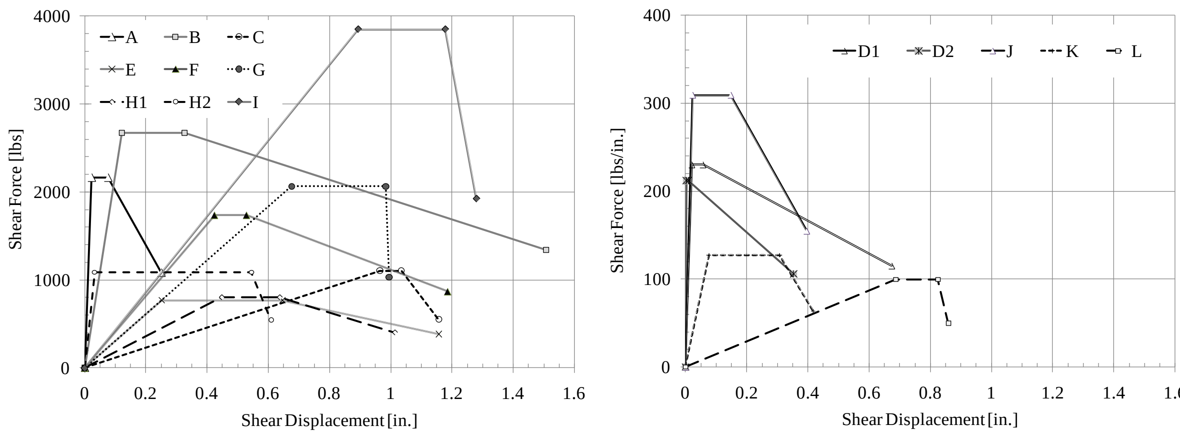


Figure 8: Backbone development

Table 4: Backbone Response (Note: 1 in. = 25.4 mm, 1 lb = 4.45 N)						
Discrete Type	*K* (lb/in.)	*V*max* (lb)	Δ*e* (in.)	Δ*y* (in.)	Δ*m* (in.)	Δ*u* (in.)
A	95500	2164	0.017	0.023	0.076	0.251
B	22243	2675	0.090	0.120	0.326	1.508
C	1145	1104	0.723	0.964	1.034	1.156
E	3058	769	0.189	0.251	0.646	1.157
F	2926	1737	0.445	0.594	0.529	1.186**
G	1855	2064	0.834	0.676	0.984	0.994**
H1	1794	803	0.336	0.448	0.637	1.014**
H2	33719	1086	0.024	0.032	0.544	0.609**
I	4304	3847	0.670	0.894	1.178	1.280**
Distributed Type	*K* ((lb/in.)/in.)	*V*max* (lb/in.)	Δ*e* (in.)	Δ*y* (in.)	Δ*m* (in.)	Δ*u* (in.)
D1	11558	230	0.015	0.020	0.057	0.673
D2	73364	212	0.002	0.003	0.009	0.353**
J	13412	309	0.017	0.023	0.148	0.397
K	1677	127	0.057	0.076	0.308	0.417
L	145	100	0.515	0.687	0.825	0.860**
* Ultimate strength from average response curve ** Displacement at 50% *V*max not measured						



a) Discrete ties b) Distributed ties

Figure 9: Shear–deformation approximation (1 in. = 25.4 mm, 1 lb = 4.45 N)

Flexural Performance Modeling with Shear Tie Data

The multi-linear shear tie performance is used to estimate the flexural response of sandwich panels. Under uniform loading, insulated sandwich panels are subjected to flexural deformation as illustrated in Figure 10. Based on the shear force diagram for a uniformly loaded panel (Figure 2) the ties located at the ends are subjected to the highest shear deformation, and the ties at the center are subjected to zero shear deformation. To properly transfer the flexural couple forces from the compression face to the tension face, adequate shear tie strength must be available. Shear tie stiffness also has a considerable influence on the panel performance. If the tie has adequate shear strength but is very flexible, compatibility will not be maintained. For this scenario, the exterior and interior wythes will resist flexure independently as two stacked plates.

The stiffness and failure mode of the shear tie can influence the ultimate flexural capacity of a sandwich panel. In a related study (Naito, Beacraft, & Hoemann, 2010), a series of sandwich wall panels were subjected to a monotonically increasing uniform load until failure occurred. The loading tree used to simulate uniform pressure on a sandwich panel and two of the panel sections tested are presented in Figure 10. The flexural response of the panel and the relative shear slip of the interior and exterior wythes were measured as illustrated in Figure 10.

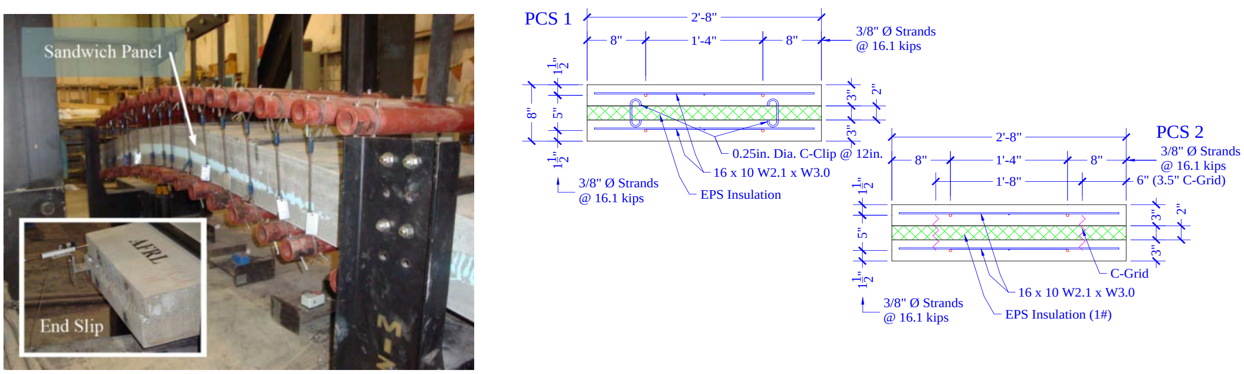


Figure 10: Uniform load evaluation of sandwich wall panels

The shear response of the ties influenced the shear failure modes of the panels. As an example, the results of three experiments are presented in Figure 11. The results include the applied pressure and midspan deflection for three panels with the same flexural design. The type of tie was varied between the panels. Panel PCS4 consisted of a flexible tie, panel PCS5 consisted of a moderately stiff tie, and PCS6 consisted of a stiff tie as illustrated in Figure 11 (right). The variation in types of ties resulted in a variation in the amount of relative slip measured between the wythes and a change in the ultimate capacity of the panels. As illustrated, the shear–deformation behavior is sensitive to the type of tie used. Between cracking and ultimate capacity, the use of a relatively stiff tie increases the strength of the panel. The type of shear tie used can significantly change the available strain energy of the wall panel.

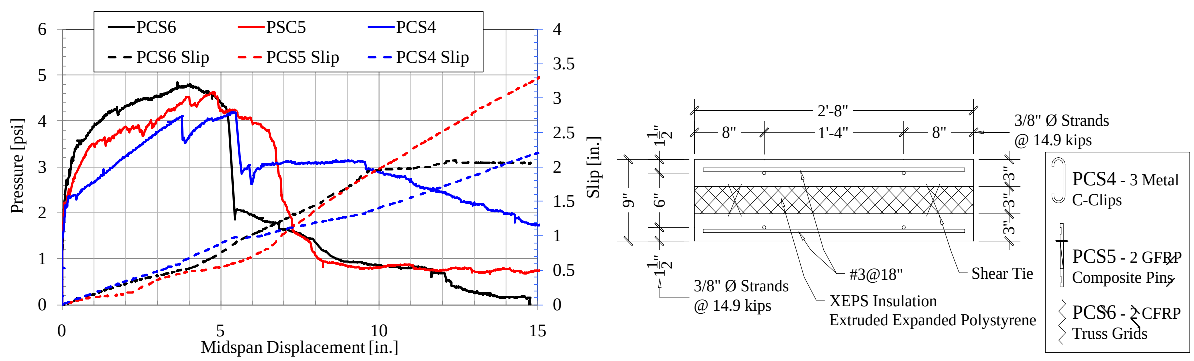


Figure 11: Flexural panel response and relative shear slip (1 in. = 25.4 mm, 1 ksi = 6.89 MPa)

Estimation of pressure-deflection response of sandwich panel

A model is presented that can be used to estimate the maximum midspan deflection of a sandwich panel subjected to a statically applied uniform pressure up to peak pressure. Prior to load application the panel is considered fully composite, and the exterior wythes act together as one system. After loading initiates, both the shear ties and core material begin to allow slip between the exterior withes, producing a reduced panel capacity or partially-composite action. In some cases, the ties fail in shear, creating non-composite action, so the wythes act as independent systems.

The model presented in Figure 12 uses the resistance functions of both the shear ties and core material to determine the level of composite action between the wythes. The foam core is assumed to have a shear strength and modulus of 0.17 and 2.8 MPa (25 and 400 psi) for EPS; 0.24 and 3.4 MPa (35 and 500 psi) for XPS. The linear stiffness of the ties is used in parallel with that of the core material to determine the level of deformation at each tie location given the shear at that location. The level of composite action is varied based on the level of slip occurring at each tie location. To determine the resistance–deformation response of the panel, a simplified approach was developed.

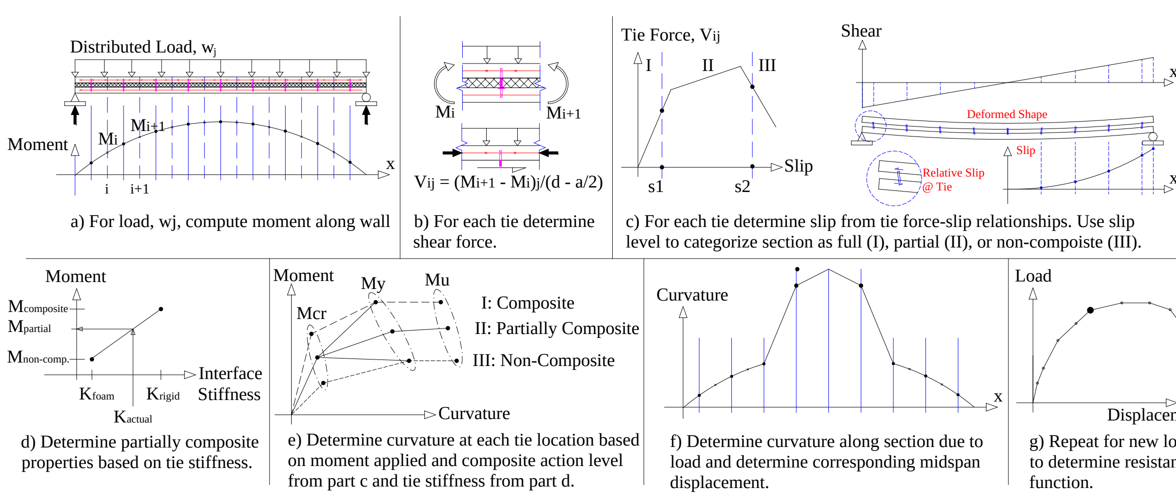


Figure 12: Resistance function methodology

In Figure 12a, an applied pressure, w_j, is applied and moment demand, M, is determined along the length of the panel. In Figure 12b, the horizontal shear demand, V_{ij}, at each tie location, i, for each pressure step, j, is estimated with the following equation.

$$V_{ij} = \frac{(M_{i+1} - M_i)_j}{d - \frac{a}{2}} \qquad \textit{Equation 5}$$

where M_{i+1} and M_i are the internal moments the ends of the current panel division, d is the depth to tensile reinforcement, and a is the depth of the rectangular stress block. For the panels tested, a is 1.27 cm (0.5 in.) on average, measured from the exterior face.

The shear demand at each tie, V_{ij}, is used along with the appropriate tie response from Figure 9 and core material response to determine the shear slip at each tie, s_{ij}. This slip is compared to two limits, s_1 and s_2, deduced from experimental results shown in Figure 9. If the slip is less than s_1, the section is considered to be fully composite. If it is greater than s_2, the section is assumed to be non-composite. If the slip is between s_1 and s_2, the panel is partially composite. The fully–partially composite slip limit, s_1, was chosen to be point of maximum shear force, taken as the midpoint between Δy and Δm in Figure 8. The partially–non-composite slip limit, s_2, is chosen to be 1.2 s_1. Beyond this limit, the ties are no longer at ultimate strength capacity.

Given the level of composite action, the curvature, φ_{ij}, at each panel division for each pressure step can be calculated. A standard section analysis was performed to determine both the fully and non-composite moment–curvature responses. A tri-linear relationship was developed for both responses using cracking, yield, and ultimate moments, calculated using standard ACI or PCI equations (depending on the type of reinforcement). These are shown as dashed lines in Figure 12e.

The partially composite moment–curvature response is determined relative to the stiffness provided by the tie and foam. A panel with foam only is assumed to be fully non-composite, while a panel containing foam and the most-rigid tie tested (D2) is assumed to be fully composite. The partially composite moment resistance of the panel at cracking, yield, and ultimate can be determined in accordance with the relationship shown in Figure 12d. From experimental observations, the partially-composite cracking moment best approximates the actual cracking moment of the panel. Therefore, the cracking moments for all three responses are set equal to that of the partially-composite response. The complete moment–curvature response of the panel according to level of composite action is shown in Figure 12e as solid lines.

With the relationships developed, the curvature for a given moment demand at a section can be determined based on the level of slip and the appropriate moment–curvature relationship. The midspan deflection for each applied pressure step j, Δ_j, is calculated using the following virtual work equation.

$$\Delta_j = \frac{L}{i} \times \Phi_j \times m_v \qquad \textit{Equation 6}$$

where L = panel length, Φ_j = curvature matrix for applied pressure step j, and m_v = virtual moment matrix for all tie locations. This form of the virtual work equation considers Φ_j and m_v the average curvatures and virtual moments over i segments of length L/i. Each Δ_j and corresponding pressure represents one point in Figure 12g.

Figure 14 shows the estimation model for one panel, experimentally tested and presented in a previous paper (Naito, Beacraft, & Hoemann, 2010). The moment–curvature information for the fully and non-composite panels is entered into the model as well as basic dimensions and quantities. The shear–deformation response of the shear tie is also entered from tests previously performed. This panel, PCS1, has a relatively very flexible steel C-clip (Figure 10). The low stiffness of this tie causes the partially-composite moment–curvature to be nearly the same as the non-composite response as shown in Figure 13. Also shown in Figure 14 are the modeled fully and non-composite responses for PCS1. The methodology, sensitive to tie type, provides a good approximation of the measured flexural response.

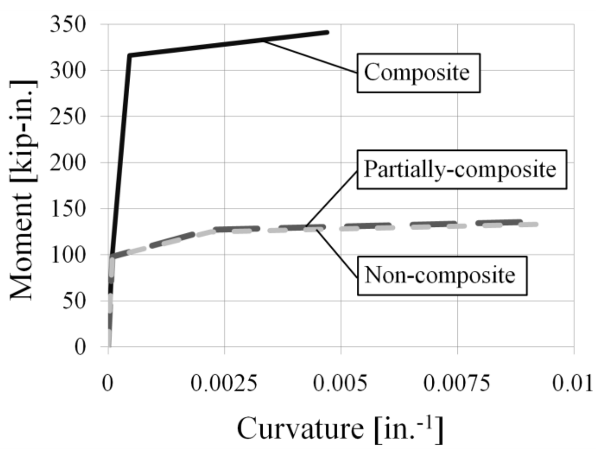


Figure 13. Moment-curvature of PCS1
(1 in. = 25.4 mm, 1 kip-in. = 0.113 kN-m)

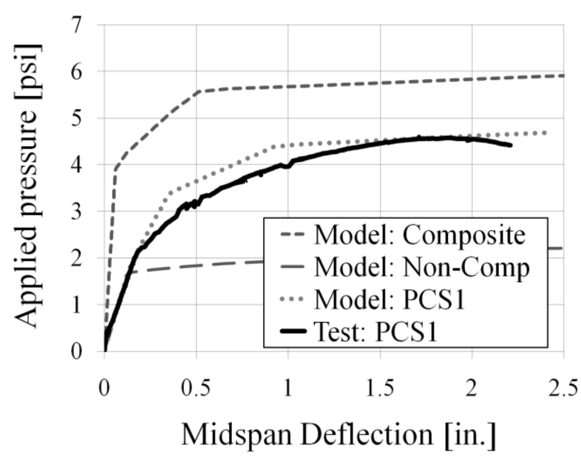


Figure 14. Model validation
(1 in. = 25.4 mm, 1 ksi = 6.89 MPa)

DISCUSSION AND CONCLUSIONS

An experimental program was conducted to examine the shear–deformation relationship of shear tie connections used for insulated precast concrete sandwich wall panels. The strengths of tie connections were measured and averaged, and simplified shear–deformation relationships were developed. The results of the study indicate that the shear performance of ties vary considerably. Average strength varies from 5.5 to 18.4 kN (1241 to 4138 lb) for discrete ties and from 17.2 to 57.8 kN/m (98 to 330 lb/in.) for distributed ties. The tie stiffness was found to be sensitive to the configuration of the tie geometry. Stiff truss type ties provided greater initial stiffness than pin type ties that work in a flexural mode. The shear strength was improved by 21% by the use of EPS insulation over XPS insulation when used in conjunction with a CFRP truss shear tie. This was attributed to the greater surface roughness of EPS.

Simplified tie resistance functions were developed based on the measured performance of the ties and were used to provide an estimate of tie slip–strength response. The tie models were used with a prediction procedure to develop an accurate estimate of the flexural response of insulated sandwich wall panels with different tie types. The use of this model indicates that the post-cracking stiffness of sandwich panels is sensitive to the type of shear tie used. Flexible ties result in lower post-cracking flexural stiffness. In all cases, however, the ultimate strength of the panel is not significantly altered.

It is important to note that the relative shear strengths alone should not be used as a measure for choosing a shear tie type. For panels designed as non-composite sections, the tension capacities of the ties are considered more crucial. Typically, the only planned occurrence for structural composite action in the life of a non-composite panel is during the construction phase, when the panels require both concrete layers to act together during the lifting phase. For non-composite panels, the interior concrete layer is usually designed to resist the in-place loads imparted to the structure. The shear strength, tension strength, stiffness, thermal conductivity, installation effort required, and cost of the tie should all be considered when determining which shear tie to use.

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