EFFECT OF NU-TIE DISTRIBUTION ON THE COMPOSITE ACTION OF PRECAST/PRESTRESSED CONCRETE SANDWICH PANELS

Xiaoyue Song, University of Nebraska—Lincoln
George Morcous, Ph.D, PE, University of Nebraska—Lincoln
Mark Lafferty, PE, Concrete Industries, Inc., Lincoln, NE
Doug Gremel, Hughes Brother, Inc., Seward, NE
Maher K. Tadros, Ph.D, PE, University of Nebraska—Lincoln

ABSTRACT

Precast/Prestressed concrete sandwich panels (PCSP) are a structurally and thermally efficient wall system for multi-story residential and commercial buildings. A typical PCSP consists of two concrete wythes separated by a layer of insulation and connected across the insulation to achieve the composite action between concrete wythes required for flexural resistance and stiffness. NU-Ties are glass fiber-reinforced polymer (GFRP) connectors that were developed by researchers at the University of Nebraska-Lincoln since the mid 1990s. The main advantages of NU-Ties are high structural performance, low thermal conductivity, and ease of installation.

This paper presents the experimental investigation carried out to determine the effect of NU-Tie distribution on the flexural capacity and stiffness of PCSP. Three 32 ft long and 5 ft wide PCSP specimens were made using three different NU-Tie distributions: a) two rows at end quarters and one row at mid quarters; b) three rows at end quarters and one row at mid quarters; and c) three rows at end quarters and two rows at mid quarters. Each specimen was tested under its own weight and gradually increasing concentrated loads at the mid-span up to failure. Test results of different specimens were compared against each other and against theoretical values calculated assuming fully composite panel. This comparison indicated that NU-Tie distribution has a significant impact on the flexural capacity and stiffness of PCSPs. The flexural capacity of a fully composite panel is attainable using proper tie distribution. In addition, analyzing the behavior of the PCSP specimens using truss models and FE models had shown better agreement with test results than using beam models.

Keywords: Sandwich panels, horizontal shear, shear connector, composite action

INTRODUCTION

Precast concrete sandwich panels (PCSP) are a structurally and thermally efficient system that is used for exterior walls in multi-story residential and commercial buildings. A typical PCSP consists of two precast/prestressed concrete wythes separated by a layer of insulation (i.e. Extruded Polystyrene [EPS) and connected across the insulation by shear connectors to achieve the composite action required for flexural resistance and stiffness. These connectors can be concrete webs or blocks, steel elements, plastic ties, or any combination of these components[1]. The low thermal resistance of steel and concrete connectors makes these products unattractive as they significantly reduce the thermal efficiency of the PCSP through thermal bridging. NU-Tie is a product developed by researchers at the University of Nebraska-Lincoln (UNL) and patented in August 15, 1995 (US Patent# 5440845). The NU-Ties are made of glass fiber-reinforced polymer (GFRP) due to the excellent thermal and mechanical properties of this material[2].

During the last decade, several research experiments were conducted to investigate the structural performance of PCSP panels using different designs of NU-Ties. This design has evolved from being a looped tie stretching in the longitudinal direction (i.e. first generation) to plane truss diagonals with various depths and angles to fit different panel thicknesses (i.e. fifth generation). Fig. 1 shows the various generations of NU-Tie design. The fifth generation of NU-Tie is different from the fourth generation in the dimensions only. Fifth generation is currently used in practical applications and, therefore, will be used in the experimental investigation presented in this paper.

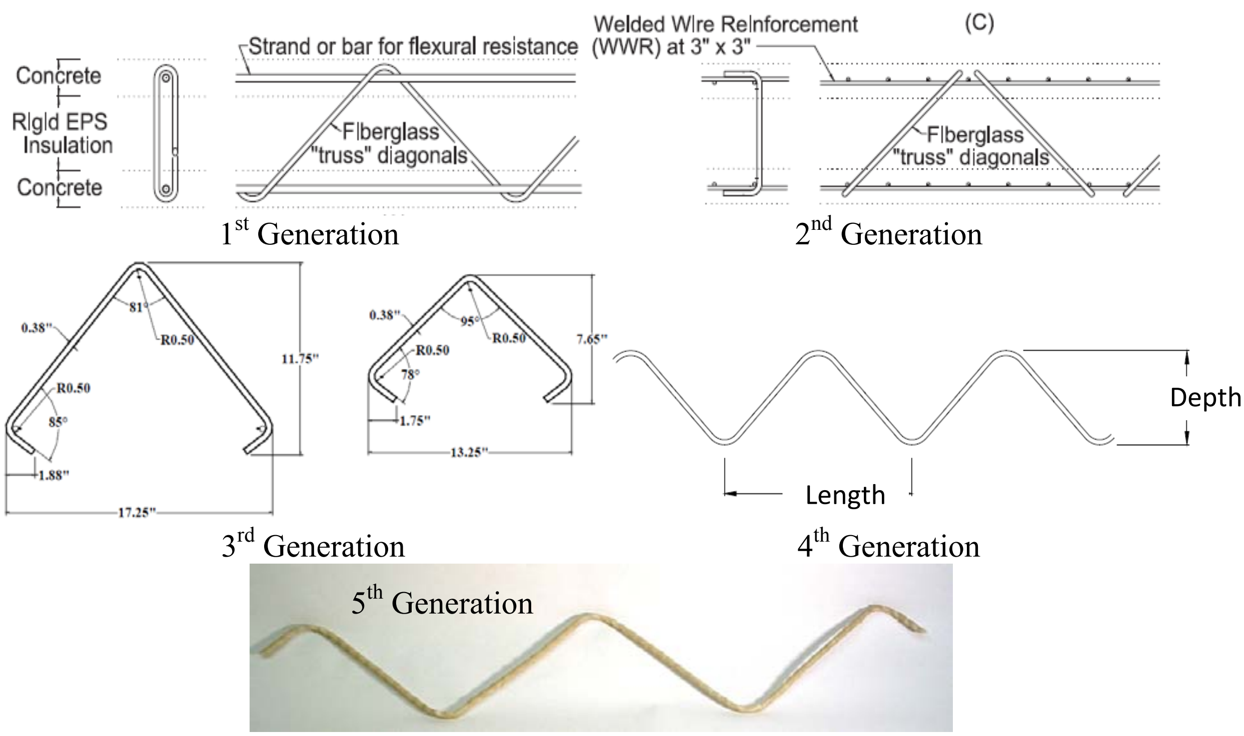

Fig. 1: NU-Tie Generations (1 in. = 25.4 mm)

All NU-Tie designs are made of 3/8 in. (9.5 mm) diameter GFRP bent rods produced by Hughes Brothers Inc., which have a cross section area of 0.11 in^2 (71 mm^2) and guaranteed tensile strength and modulus of elasticity of 110 ksi (759 MPa) and 5920 ksi (40,848 MPa), respectively. The tensile strength testing of GFRP rods was performed according to the Guide Test Methods for Fiber Reinforced Polymers for Reinforcing or Strengthening Concrete Structures prepared by ACI Subcommittee 440[3]. Fig. 2 plots the average stress-strain diagram of testing six specimens. The test results show that the average tensile strength is 122.7 ksi (847 MPa), and average modulus of elasticity is 5,980 ksi (41,262 MPa).

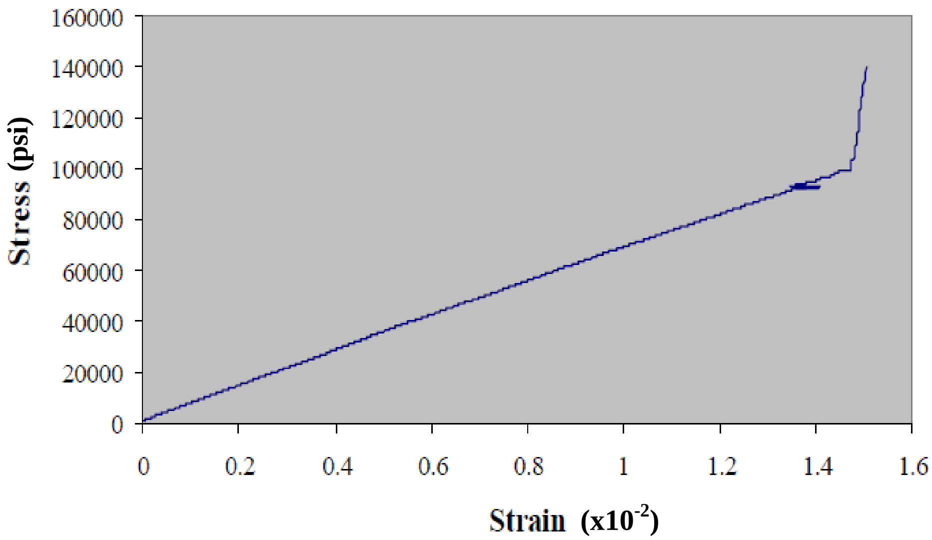

Fig. 2: Stress-Strain Diagram of GFRP Bars

PCSP can be designed as non-composite, semi-composite, and composite panels[4]. For non-composite panels, the flexural capacity is that of individual solid panels, each has the sectional properties of a concrete wythe. Connectors are used to keep the wythes together and the panel intact during handling. Deflections are calculated using the sum of wythe flexural stiffness. Semi-composite panels are those panels that behave as composite and non-composite at different times and under different loading conditions. For composite panels, the flexural capacity is that of a solid panel that has the same cross section of the two/three wythes. Shear connectors are used to transfer horizontal shear forces between wythes. This force is calculated using the strength method given in the PCI Design Handbook, 6th Edition (2004) Section 4.3.5 “Horizontal Shear Transfer in Composite Members”[5]. In this method, the horizontal shear force is taken as the lesser of the maximum compressive force in concrete and maximum tensile force in the reinforcement/prestressing. This force is then used to determine the required number of shear connectors over the horizontal shear span, which is one-half the clear span for simply supported panels. Most manufacturers of shear connectors use the same method to determine the amount of shear connectors for composite panels and distribute these connectors uniformly along the horizontal shear span. Despite the simplicity of this method, it does not provide the designer with optimized distribution of shear connectors or evaluate the impact of connector distribution on the flexural capacity of the PCSP. In addition, deflection of composite panels is commonly calculated using the beam model, which does not consider the effect of either the number or the distribution of shear connectors. An arbitrary percentage of the fully composite section is commonly used to account for the loss in beam stiffness, which is inaccurate and requires calibration[6].

The objective of this study is to experimentally investigate the effect of NU-Tie distribution on the behavior of PCSP and evaluate the accuracy of the truss model and FE model versus beam model for PCSP deflection calculations. The paper is organized as follows: The second section presents the design and production of the test specimens. The third section presents the setup and procedures for testing the three specimens with different NU-Tie distributions. The fourth presents test results and data analysis as well as the analytical models developed for deflection prediction. The last section summarizes the main conclusions and future recommendations

PRODUCTION OF TEST SPECIMENS

The three specimens are full scale PCSPs that are 32 ft (9.76 m) long 5 ft (1.53 m) wide and 10 in. (254 mm) thick each. Each panel has two 3 in. (76 mm) thick concrete wythes and a 4 in. (102 mm) thick EPS layer. The specimens have identical longitudinal and transverse reinforcement as shown in Fig. 3. Longitudinal reinforcement consists of 3-7/16 in. (11.1 mm) diameter Grade 270 low-relaxation strands in each wythe tensioned at 0.7 f_{pu}. Transverse reinforcement consists of 3/8 in. (9.5 mm) diameter bars at 32 in. (813 mm) spacing in each wythe.

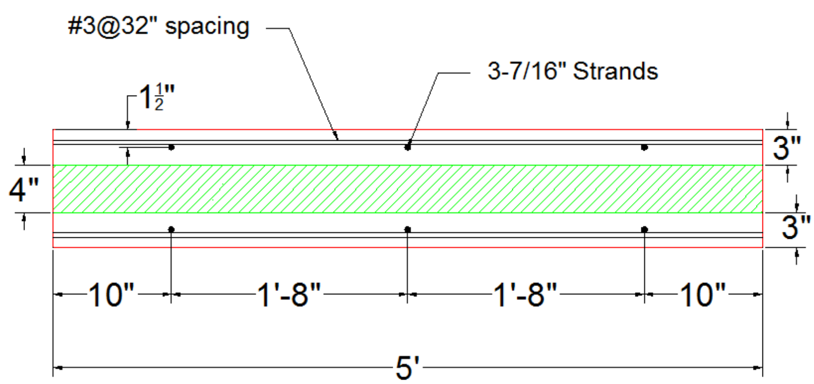

Fig. 3: Section view of the test specimens

The three specimens have different number and distribution of NU-Ties as shown in Fig. 4. The first panel has two ties in the end quarters, and one tie in the middle section (Panel 2-1). The second panel has three tie rows in the end quarters and one tie row in the middle section (Panel 3-1). The third panel has three tie rows in the end quarters and two tie rows in the middle section (Panel 3-2). The reason for having these three designs is to evaluate the effect of number of ties in the end quarters versus that of number of ties in the middle half on the flexural capacity and stiffness of the PCSP. The three test specimens were produced by Concrete Industries Inc., Lincoln, NE, on November 25th, 2008. The fabrication process involved the following steps:

1. Setup the forms and lubricate the bed for concrete placement
2. Stress the strands and place the reinforcement and attachments of the bottom wythe
3. Place the concrete of the bottom wythe.
4. Prepare the EPS insulation panels with the NU-Ties as shown in Fig. 5
5. Place EPS panels with NU-Ties on the fresh concrete of the bottom wythe
6. Stress the strands and place the reinforcement and attachments of the top wythe
7. Place the concrete of the top wythe.
8. Cover and cure the panels for 16 hours.
9. Release the strands and move the panels to storage using suction lifting equipment

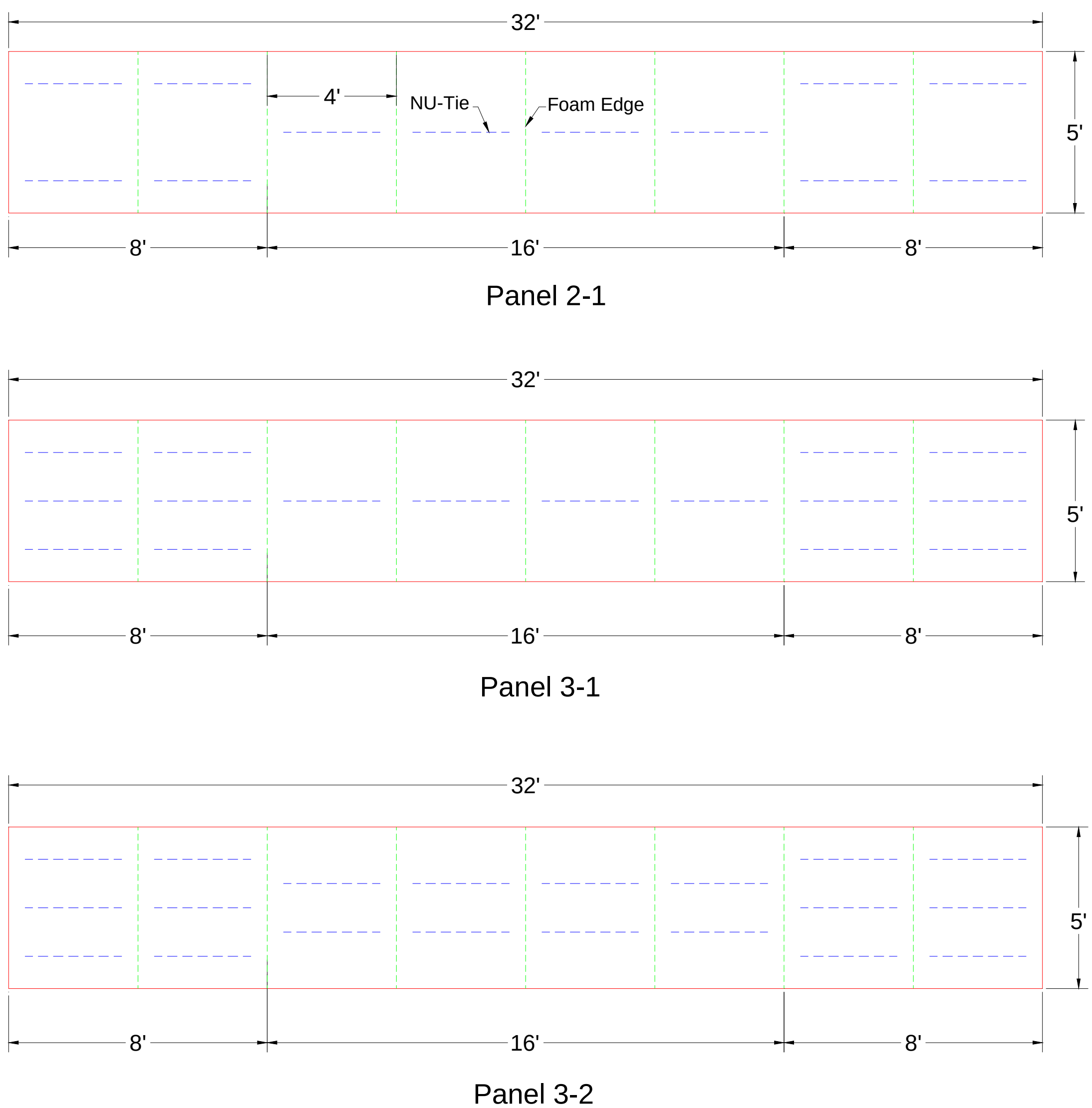

Fig. 4: Plan view of the three test specimens

Fig. 5: Inserting NU-Ties in the slots of the EPS panels

TESTING SETUP AND PROCEEDURES

The test was held at Concrete Industries Inc., Lincoln, NE, on January 9th, 2009. Six strain gauges and two potentiometers were installed on each panel. Three strain gauges were installed on the top surface and three strain gauges were installed on the bottom surface along the centerline of the panels as shown in Fig. 6. The notations of the strain gauges were "E", "C", and "W" to represent "East", "Center", and "West" respectively; and "1"and "2" to represent "Top" and "Bottom" respectively. For example, "E1" means the strain gauge installed on the top surface at the east side of the panel. Two potentiometers were installed on the bottom surface at middle of the panels to measure the mid-point deflections.

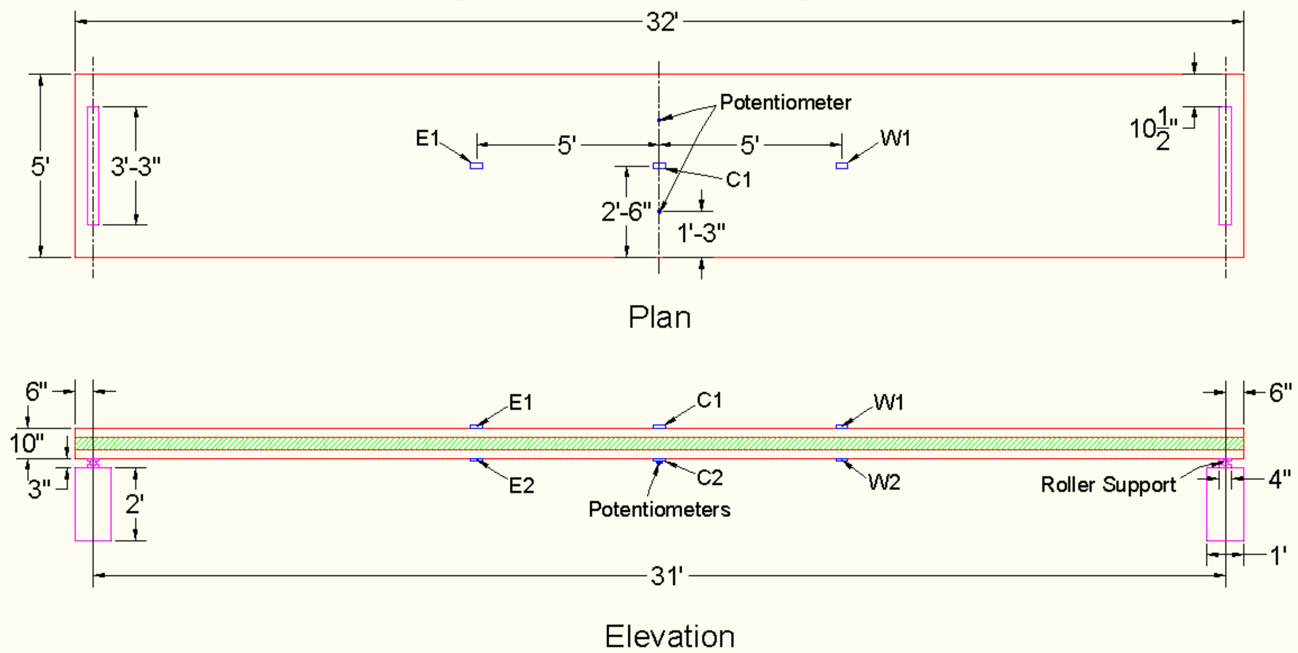

Fig. 6: Testing setup

Test setup and procedures are identical for the three specimens and summarized as follows:

1. Two roller supports were placed 31 ft (9.5 m) apart and two hydraulic jacks were placed 9 ft (2.75 m) apart and 11 ft (3.36 m) away from the support as shown in Fig. 7.
2. Each panel was placed on the fours supports using suction lifting equipment.
3. Middle jacks were adjusted, so that the four supports are aligned and the panel is leveled.
4. Strain gauges and the potentiometers were attached as top and bottom surfaces, as shown in Fig. 6, and connected to the data acquisition system.
5. Strain gauges and the potentiometers were balanced to have zero initial reading.
6. The middle hydraulic jacks were released one at a time to let the panel deflect under its own weight. Strains and deflections were recorded and the panel was inspected for cracking and/or delamination between the concrete and EPS layers.
7. Panels were slowly loaded using prefabricated concrete blocks that are 13 in. (330 mm) x 13 in. (330 mm) x 13 in. (330 mm) each, which weighs slightly less than 200 lbs (90 kg). The blocks were numbered and the exact weight of each block was determined using a scale before loading. Fig. 8 shows the loading sequence for each panel and the location of the loading blocks.
8. Strains and deflections were recorded after each loading and the panel was inspected for cracking and/or delamination between concrete and EPS layers.
9. Loading continued up to failure for panel 2-1. However, panels 3-1 and panel 3-2 were able to carry all the 21 concrete blocks without any signs of failure. Therefore, these two panels were also loaded using a 400 lbs (182 kg) reinforced concrete corbel and a 225 lbs (102 kg) steel plate as shown in Fig. 9. These loads remained on the panels for 10 mins and then, the test was stopped for safety concerns and the panels were unloaded.

Fig. 7: End roller supports and temporary middle supports of the test specimen

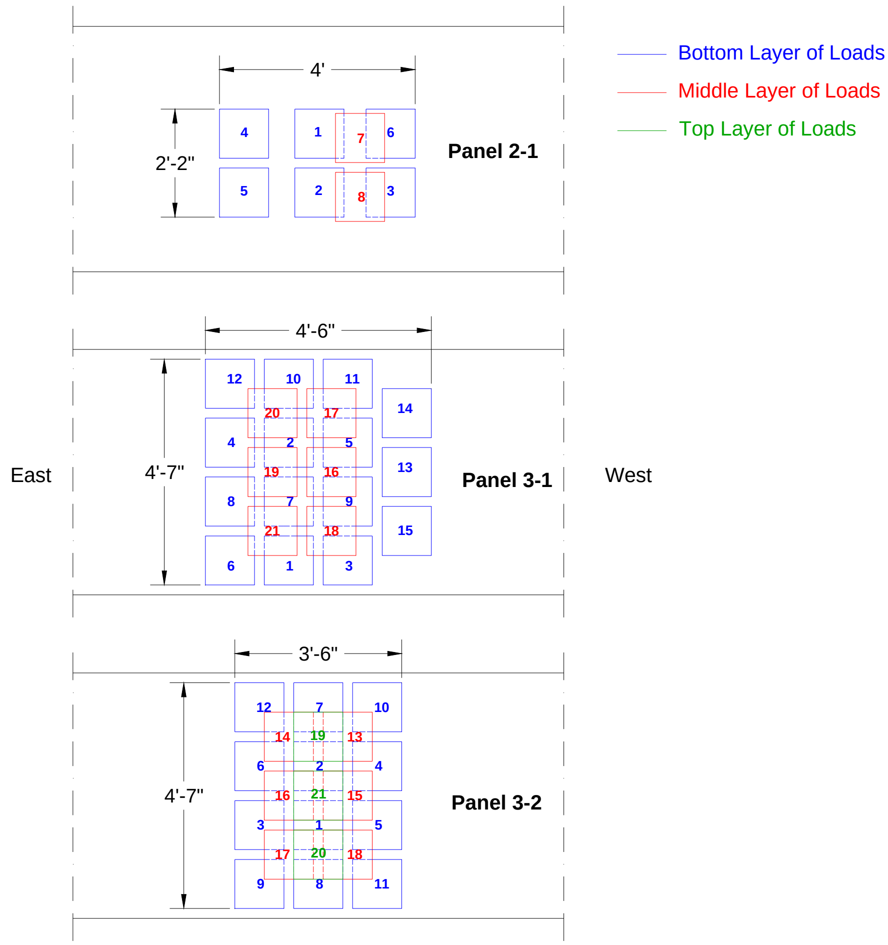

Fig. 8: The location and sequence of loading the three specimens

Fig. 9: Loading of Panel 3-2

TEST RESULTS AND DATA ANALYSIS

Fig. 10 plots the load-deflection relationships of the three panels. The deflection values at zero load represent the deflection under the panel own weight only, which are 2.5 in. (63.5 mm), 1.2 in. (30.5 mm), and 0.9 in. (22.9 mm) for the panels 2-1, 3-1, and 3-2 respectively. Fig. 10 indicates that the load-deflection relationships are linear up to the cracking loads, which are approximately, 1,184 lb (5.3 kN) for panel 2-1, 2,564 lb (11.4 kN) for panel 3-1, and 2,358 lb (10.5 kN) for panel 3-2. Non-linear relationship continues up to the ultimate loads, which are approximately 1,579 lb (7 kN) for panel 2-1, and 4,754 lb (21.1 kN) for panels 3-1 and 3-2. The ultimate load of panel 2-1 is the load that caused the panel to fail, while the ultimate loads of panels 3-1 and 3-2 are the maximum loads the panels were subjected to. Although these loads did not cause the specimens to fail, the test was stopped for safety concerns. These loads will be used to conservatively estimate the actual flexural capacity of the specimens. It should be mentioned that load-deflection relationships should not be interpreted or compared without considering that both weight and location of loading blocks are not identical in all tests.

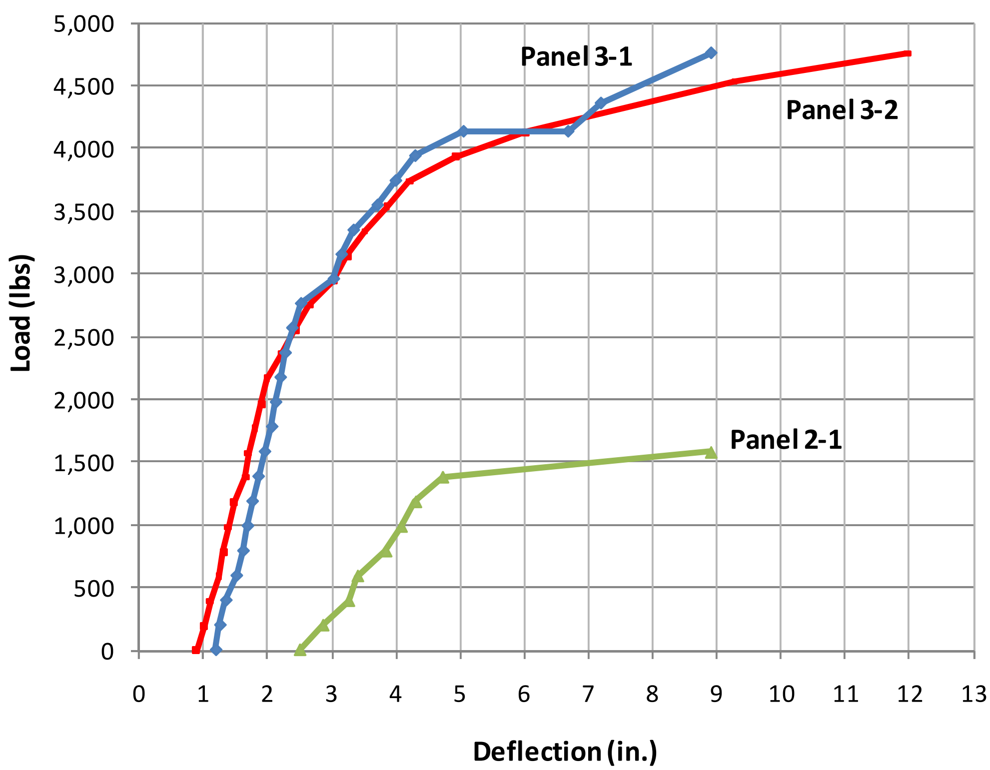

Fig. 10: Load-deflection relationship of the three panels (1 in. = 25.4 mm; 1000 lb = 4.448 kN)

Fig. 11 plots the compressive strength of the concrete used in the fabrication of top and bottom wythe against age. The last point in this plot represents the compressive strength of the concrete at the time of testing. For strength and prestress loss calculations, the following values of the concrete strength were used: 3,500 psi (24.2 MPa) at release, and 9,500 (66.2 MPa) at final. Prestress loss calculations were performed according to the 6th Edition of the PCI Design Handbook (2004), which resulted in a total prestress loss of approximately 9%. The nominal flexural capacity of the panel section (ΦM_n) was calculated using strain compatibility and assuming a fully composite section and a resistance factor (Φ) of 1.0. This resulted in a theoretical capacity of 74 kip.ft (100.4 kN.m.), depth of compression block of 0.37 in. (9.4 mm), and ultimate stress in prestressing strands of 270 ksi (1863 MPa).

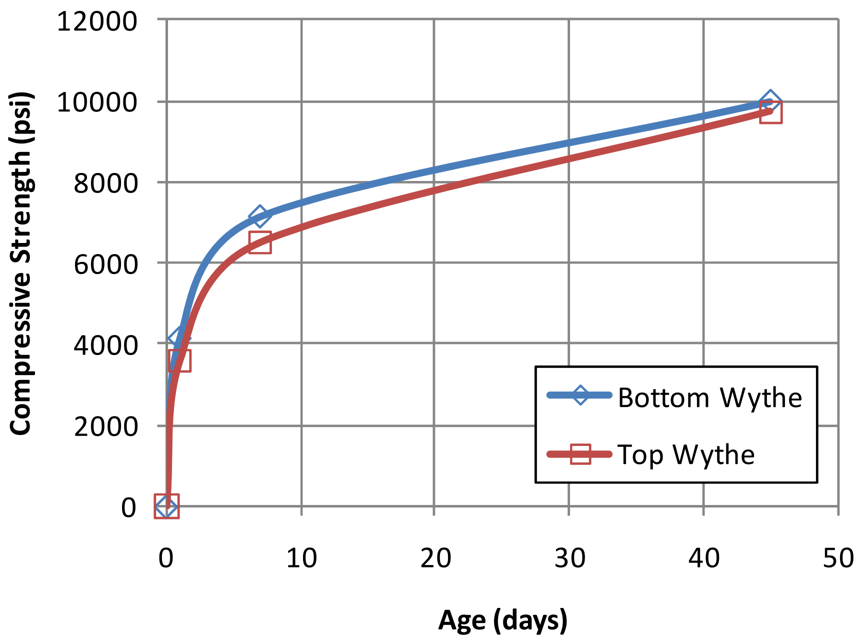

Fig. 11: Concrete compressive strength against age (1000 psi = 6.9 MPa)

Table 1 compares the theoretical flexural capacity of each specimen with its actual flexural capacity obtained from testing. The ratios of actual-to-theoretical capacity indicate that the NU-Tie distribution in panel 2-1 results in an actual flexural capacity that is 76% of the theoretical capacity of a fully composite section. This reduction in the flexural capacity is primarily due to inadequate number and distribution of NU-Ties, which resulted in horizontal shear failure as shown in Fig. 12. The ratios of actual-to-theoretical capacity in Table 1 also indicate that NU-Ties in panel 3-1 and panel 3-2 had superior performance to those in panel 2-1. The number of NU-Ties in those two panels was calculated using the strength method presented in the PCI Design Handbook, 6th Edition (2004) section 4.3.5. However, the distribution of these ties was not uniform but triangular similar to the horizontal shear diagram. This distribution resulted in an actual flexural capacity that exceeds the theoretical capacity of a fully composite section. It should be noted that although the ultimate capacity of the two specimens was exceeded in testing, the two specimens did not fail. This might be because the actual ultimate strength of the used prestressing strands was higher than the specified value, which is 270 ksi (1863 MPa).

Table 1: Comparing the Theoretical against Actual Flexural Capacity of Test Specimens

Panel	L (in)	M_n (kip.in)	W_{ow} (kip/in)	M_{ow} (kip.in)	P_u (kip)	a (in)	M_u (kip.in)	$M_{u\text{-}actual}$ (kip.in)	$M_{u\text{-}actual}$/ M_n
2-1	372	**888**	0.031	540.6	1.579	48	137.4	**678**	**0.76**
3-1	372	**888**	0.031	540.6	4.754	54	410.0	**951**	**1.07**
3-2	372	**888**	0.031	540.6	4.754	42	417.2	**958**	**1.08**

Fig. 12: Movement of top wythe relative to bottom wythe in Panel 2-1

Table 2 shows the calculation of the modulus of rupture (MOR) of the panel section based on the cracking loads obtained from testing. Gross section properties of a fully composite section are used to calculate the tensile stress at the extreme bottom fibers due to the panel's own weight and applied loads. Table 2 shows that the calculated MOR is approximately 35% lower than the ACI 318-08 recommended value of 7.5√fc' for panels 3-1 and 3-2 and 50% lower than the recommended value for panel 2-1. This indicate that the number and distribution of NU-Ties in panels 3-1 and 3-2 results in better composite action and, consequently, higher cracking load than that of panel 2-1.

Table 2: Calculation of MOR of the Test Specimens

Panel	L (in)	W_{ow} (kip/in)	M_{ow} (kip.in)	P_{cr} (kip)	a (in)	M_{cr} (kip.in)	$M_{cr\text{-}actual}$ (kip.in)	P (kip)	A_g (ksi)	S_g (in^4)	MOR (ksi)	Coeff. $\sqrt{f_c'}$
2-1	372	0.031	540.6	1.184	48	103.0	644	119	360	936	-0.36	3.7
3-1	372	0.031	540.6	2.564	54	221.1	762	119	360	936	-0.48	5.0
3-2	372	0.031	540.6	2.358	42	206.9	747	119	360	936	-0.47	4.8
										Average	-0.44	4.5

ANALYTICAL MODELS

The structural analysis of a PCSP as a simply supported beam with a reduced moment of inertia for partial composite panel does not represent the true behavior of the panel. This is because the beam model cannot explicitly account for the impact of NU-Tie distribution on the stiffness of the panel. In order to predict the behavior of PCSPs with different number and distribution of NU-Ties, two modeling methods are investigated. The first method is developing planar truss models in which the top chord members represent the top wythe, bottom chord members represent the bottom wythe, and diagonal members represent tie legs. Fig. 13 shows the three planar truss models developed for the three specimens with different NU-Tie distributions. In each model, truss elements are assumed to be located at the centerlines of actual elements and have the equivalent section properties. For example, the geometric properties of a diagonal member in the end quarter of the panel 3-1 are equal to three times the geometric properties of one tie leg. Connections between the diagonal members and top and bottom chord members are assumed to be pinned with rigid end zone equal to the portion of tie leg embedded in concrete. The three models are assumed to be simply supported and subjected to a uniform load that represents their own weight (75 psf). Analysis results of the three truss models are listed in Table 3.

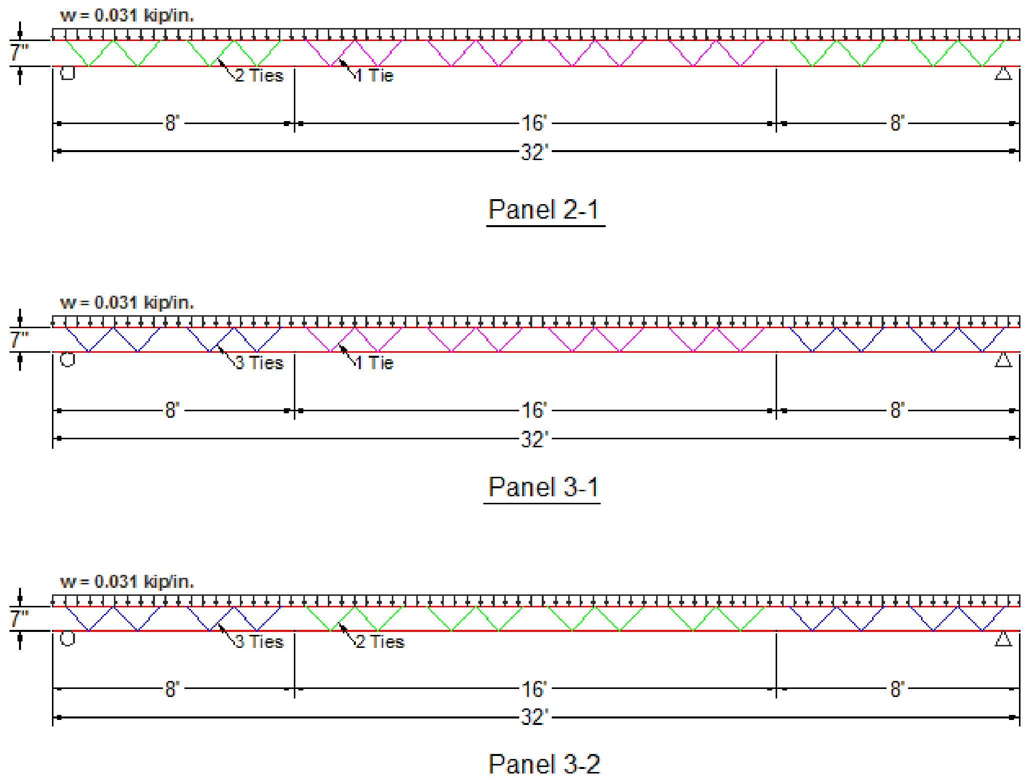

Fig. 13: Truss models of the three test specimens

The second modeling method is developing three dimensional FE models in which the top and bottom wythes are modeled as shell elements, and tie legs are modeled as frame elements. Fig. 14 shows the model developed for the panel 3-1. In each model, shell and frame elements are assumed to be located at the centerlines of actual elements and have their exact section properties. Connections between the frame and shell elements are assumed to be pinned with rigid end zone equal to the portion of tie leg embedded in concrete. The three models are assumed to be simply supported and subjected to a uniform load that represents their own weight (75 psf). Fig. 15 shows the vertical deflection of panel 3-1 under its own weight. Analysis results of the three FE models are listed in Table 3.

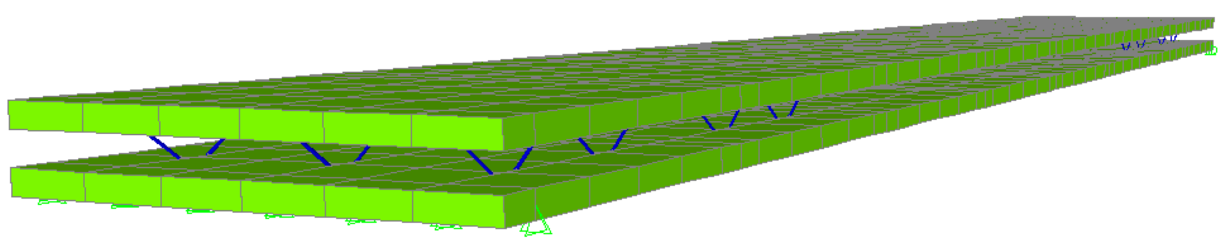

Fig. 14: 3D FE model of the panel 3-1

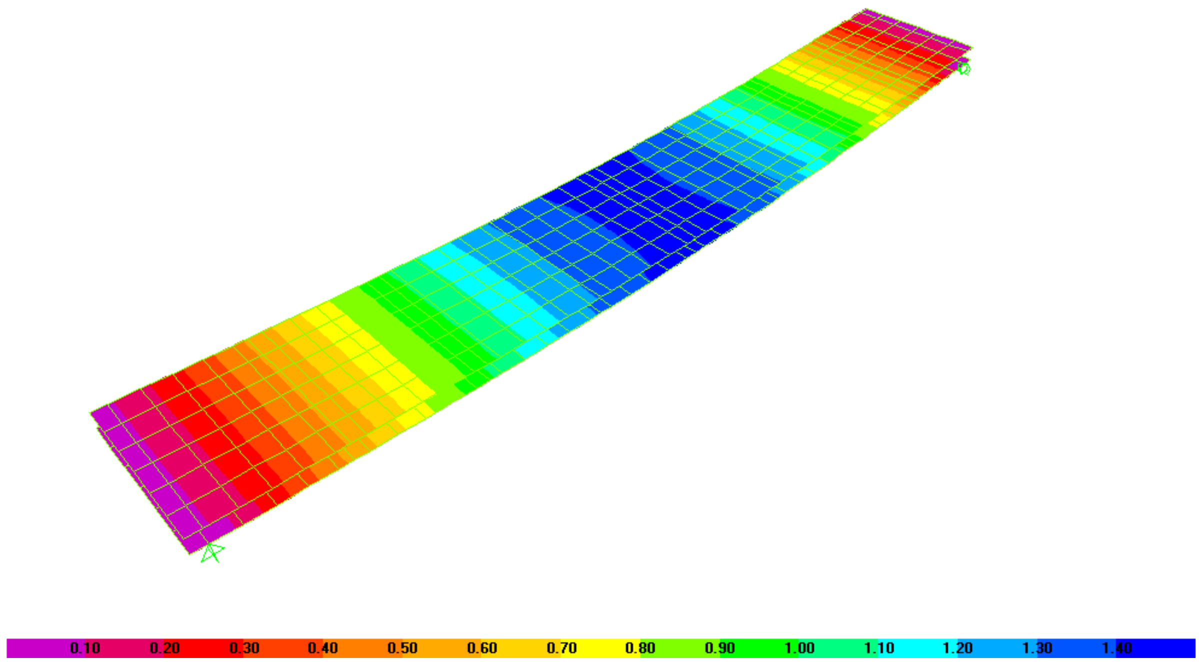

Fig. 15: Deflection of the 3-1 panel estimated using the 3D FE model

Table 3: Comparing Theoretical Deflections Calculated Using Different Models against Actual Deflections

Panel	L (in)	E (ksi)	I_g (in^4)	W_{ow} (kip/in)	D_{beam} (in)	D_{truss} (in)	D_{FE} (in)	D_{actual}* (in)	D_{actual}/D_{beam}	D_{actual}/D_{truss}	D_{actual}/D_{FE}
2-1	372	5,556	4,680	0.031	0.30	1.70	1.75	2.5	**8.3**	**1.5**	**1.4**
3-1	372	5,556	4,680	0.031	0.30	1.40	1.42	1.2	**4.0**	**0.9**	**0.8**
3-2	372	5,556	4,680	0.031	0.30	1.30	1.33	0.9	**3.0**	**0.7**	**0.7**

* Measured values are slightly less than actual deflections because of panel saging between intermediate supports

Table 3 presents the theoretical deflections of the three specimens under their own weight as estimated by three different models: 1) beam model, 2) truss model; and 3) FE model. Comparing these values against the actual deflections measured during testing indicates that the beam model with gross section properties highly underestimates the panel deflection. Significant reduction in the moment of inertia of the panel has to be assumed to account for the loss in panel stiffness due to partial composite behavior. This reduction is not constant as it varies significantly based on the number and distribution of the NU-Ties. For example, panel 2-1 would have a reduction factor of 0.12, while panel 3-2 would have a reduction factor of 0.33. Table 3 also indicates that both planar truss models and FE models provide much more reasonable estimates of panel defections under uniform load than the beam models. The difference between the estimated and actual deflections of panels 3-1 and 3-2 is mainly due to the fact that measured deflections are slightly less than actual deflections because of panel sagging under its own weight between intermediate supports (refer to Fig. 7). Also, it can be concluded that the planar truss models and FE models provide very close results; therefore, planar truss model is highly recommended for its greater relative simplicity and computational efficiency.

CONCLUSIONS

Based on the experimental and analytical investigations presented in this paper, the following conclusions are made:

1. The number of NU-Ties required to achieve full composite action should be calculated using the PCI Design Handbook method for horizontal shear in composite members. However, a triangular distribution of the horizontal shear along the shear span should be used to determine the most efficient distribution of NU-Ties. The number of ties in the end quarters can be used along the entire panel to simplify panel fabrication.
2. The specimen with less number of NU-Ties than required at the end quarter (Panel 2-1) had a premature failure as its ultimate flexural capacity was only 76% of the theoretical capacity of fully composite section calculated using strain compatibility. This specimen had also the lowest MOR and highest deflection that the theoretical values calculated using gross section properties.
3. The specimens with the required number of NU-Ties at the end quarters (Panels 3-1 and 3-2) had similar structural performance regardless of the number of NU-Ties at the middle section. The ultimate flexural capacity of these two specimens exceeded the theoretical capacity of fully composite section calculated using strain compatibility. Also, the two specimens had MOR and deflection values that are very close to theoretical ones. This indicates that the number of NU-Ties in the middle portion of the specimen has minimal impact on its structural behavior.
4. Calculating deflections of PCSP using the truss models and FE models results in consistent and more realistic deflection predictions than those calculated using the beam model and gross section properties. Truss models provide comparable predictions to those obtained from FE models while being relatively simpler and computationally more efficient. However, it should be noted that both FE analysis and truss analysis require a computer program and cannot be easily performed manually as the beam analysis.

NOTATION

L	Span, in.
a	Distance of load application, in.
$W_{o.w.}$	Own weight, kip/in.
$M_{o.w.}$	Moment due to own weight, kip.in.

P_u	Ultimate Load, kip
M_u	Moment due to ultimate load, kip.in.
P_{cr}	Cracking Load, kip
M_{cr}	Moment due to cracking load, kip.in.
D_{actual}	Actual deflection, in.
D_{truss}	Theoretical deflection calculated using truss model, in.
D_{FE}	Theoretical deflection calculated using FE model, in.
D_{beam}	Theoretical deflection calculated using beam model, in.
$M_{u\text{-}actual}$	Actual ultimate moment, kip.in.
M_n	Theoretical ultimate moment assuming full composite, kip.in.
$M_{cr\text{-}actual}$	Actual cracking moment, kip.in.
E	Modulus of elasticity, ksi
MOR	Modulus of Rupture, ksi
P	Effective prestressing, kip
A_g	Gross Area, in^2
I_g	Gross moment of inertia, in^4

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APPENDIX

Calculation of horizontal shear for precast concrete sandwich wall panels using NU-Tie

By using the given parameters, calculate the NU-Ties needed for horizontal shear of panel. (Parameters as for Panel 3-1)

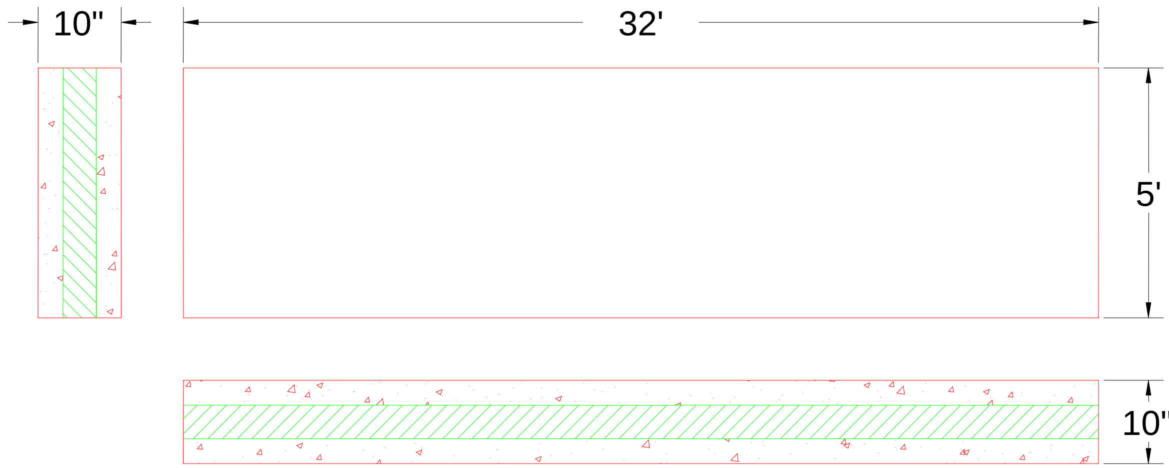

Fig. A1: Precast sandwich wall panel for NU-Tie design (Not-to-scale)

Panel properties:

Panel Width (B)	=	5	ft
Panel Total Length (Lt)	=	32	ft
Panel Span (L)	=	31	ft
Inside Wythe Thickness (t_i)	=	3	in
EPS Thickness (t_e)	=	4	in
Outside Wythe Thickness (t_o)	=	3	in
Total Panel Thickness (T)	=	10	in

NU-Tie properties:

Tie Cross Section Area (A_b)	=	0.11	in^2
Tie Angle (α)	=	44.0	deg.
Tie Depth (d)	=	8.00	ft
Tie Tensile Strength (f_u)	=	110	ksi

Design assumptions:

Ultimate Load (W_u)	=	106	psf
Exposure Factor (C_e)	=	0.70	
Strength Reduction Factor (C_r)	=	0.50	
Resistance Factor (φ)	=	1.00	

Solution:

Ultimate Moment (M_u) $= \dfrac{W_u \times B \times L^2}{8} = \dfrac{106 \times 5 \times 31^2}{8 \times 1000} = 63.7$ kip.ft

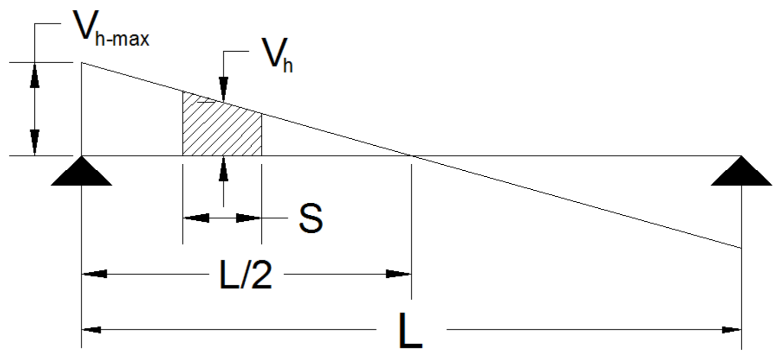

Fig. A2: Shear distribution of panel

Total Horizontal Shear ($V_{h\text{-total}}$) $= \dfrac{M_u \times 12}{8} = \dfrac{63.7 \times 12}{8} = 95.5$ kip

Maximum Horizontal Shear ($V_{h\text{-max}}$) $= \dfrac{V_{h-total} \times 4}{L} = \dfrac{95.5 \times 4}{31} = 12.32$ kip/ft

Horizontal Shear Gradient (G) $= \dfrac{V_{h-\max} \times 2}{L} = \dfrac{12.32 \times 2}{31} = 0.80$ kip/ft per ft

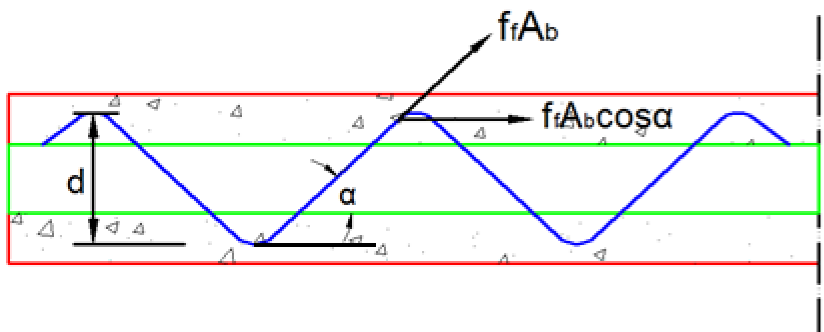

Fig. A3: NU-Tie strength calcualtion

Factored Strength (f_f) = $\phi \times f_u \times C_e \times C_r = 1.00 \times 110 \times 0.7 \times 0.5 = 38.50$ ksi

Leg Capacity (F) = $A_b \times f_f \times \cos\alpha = 0.11 \times 38.50 \times \cos 44^o = 3.05$ kip

Since the length of NU-Tie is approximately 4 ft, the panel can be divided into segments that are multiples of 4 ft in length. In this example, 8 ft long segments are considered to reduce the number of variations in NU-Ties distribution in the 32 ft long panel. The required number of legs from the panel end to 8 ft along the panel (segment #1), is calculated as follows:

$$\text{Number of Legs Needed (N)} = \frac{(V_{h-\max} - G \times (4-0.5)) \times (8-0.5)}{F}$$

$$= \frac{(12.3 - 0.8 \times (4-0.5)) \times (8-0.5)}{3.05}$$

$$= 23.5 \text{ legs}$$

Since one NU-Tie contains 4 legs,

Number of NU-Tie needed $= 23.5 \div 4 = 5.9$ Use 6 NU-Ties for segment #1.

A similar procedure was followed to determine the number of ties needed for segment #2 (i.e. the next 8 ft). This resulted in 8 legs, which is equal to 2 NU-Ties.