Behavior and Design of Composite Precast Prestressed Concrete Sandwich Panels with NU-Tie

by

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ABSTRACT

A precast concrete sandwich panel system with a high thermal resistance and optimum structural performance was developed in the early 1990s and patented by the University of Nebraska-Lincoln. It combined the high structural efficiency and thermal insulation capacity through use of a special fiberglass composite "truss" connector, called the NU-Tie. Previous state of the art work had used either thermally conductive steel connectors or structurally non-composite polymer connectors for thermal efficiency. The new Fiber-reinforced polymer (FRP) NU-Tie connector can provide for both effects simultaneously. The connector configuration was revised over time to allow for the most cost-effective manufacturing and application. It started as looped truss, and ended as a plane truss. The purpose of this report is to experimentally verify the structural efficiency of the recent plane truss versions of the NU-Tie and to demonstrate how to conduct the structural design of a composite sandwich panel. Recommended modifications in the Precast/Prestressed Concrete Institute handbook (PCI Design Handbook) method of design of sandwich panels will be illustrated.

INTRODUCTION

Typical precast concrete sandwich panel (PCSP) systems are composed of two concrete wythes with insulation placed in a third wythe between the two concrete wythes. The concrete wythes are generally connected through the insulation using metal, concrete, polymer (plastic), or glass fiber reinforced polymer (GFRP) connectors. PCSP systems can be designed to be composite or non-composite structural members. In non-composite walls, one wythe is counted on to resist the entire applied loading, and the second wythe is considered to be non-structural. In composite construction, the two concrete wythes share in the load resistance through the connectors that are capable of resisting the interface shear force resulting from composite action. PCSP systems can be classified into three major categories:

1. Fully composite panels
2. Non-composite panels
3. Partially composite panels

Full composite action is seldom achieved unless the middle wythe is penetrated with a significant of concrete. In this situation, the thermal efficiency is significantly compromised. It has been shown through earlier studies that a 2% penetration of the insulation wythe with a conductive material, such as steel or concrete, could result in a 40 percent loss in thermal efficiency, see Ref. 5.

A conventional approach to achieving significant (not full) composite action is through use of a welded wire girder connector (WWG). This is the most popular system as of the writing of this report. The only disadvantage is the loss of thermal efficiency. Thermal efficiency comes from using GFRP or plastic connectors. However, prior to the NU-Tie development no product was available on the market to combine the thermal efficiency with structural composite action demands.

EVOULTION OF THE NU-Tie SHAPE

The NU-Tie is a patented product, covered by U.S. patent number 5,440,845, dated August 15, 1995. The original patent described the concept and the various shapes, the NU-Tie could take. Research that followed has attempted to revise the shape in an effort to optimize two processes: the connector manufacturing and concrete sandwich panel construction, whether cast-in-place or precast concrete construction. Evolution of the shape development is represented in Fig. 1 to Fig. 9.

The first version was NU-Tie-V1; it was in a shape of looped tie that is stretched in the longitudinal direction to form truss diagonals, as in Fig.1. This was individually made for the University testing reported in References 5, 6, 8 and 10.

The next generation was NU-Tie-V2; it was a C-shape single connector (CSS) as shown in Fig.2. It was created as two halves of a closed tie system available at the company producing the ties. This shape was used on the precast concrete house in Omaha, Nebraska, covered in Ref. 11.

The third generation was NU-Tie-V3; a V-Shape Single connector (VSS) as shown in Fig.3 – Fig.5. This product was also created from a closed tie configuration, and was used in the first phase of testing reported in this report.

The next-to-last version tie, the NU-Tie-V4, is shaped as a series of truss diagonals. It is a plane truss as shown in Fig. 6. It required new GFRP plant equipment which was purchased as a result of the previous success with the experimental projects.

Upon completion of the final testing reported herein, Concrete Industries, Inc. of Lincoln, Nebraska, became interested in introducing this system in their sandwich panel product line. They, jointly with UNL researchers and Hughes Bros. of Seward, NE decided on the final shape to use in precast concrete panel production. That shape is represented by Fig. 7 - Fig.9 for various panel/insulation thicknesses. The only difference between this final shape and the one subjected to the last series of tests is in the ending tips of the truss. The NU-Tie-V5 is relatively easy to place in slots in the insulation board.

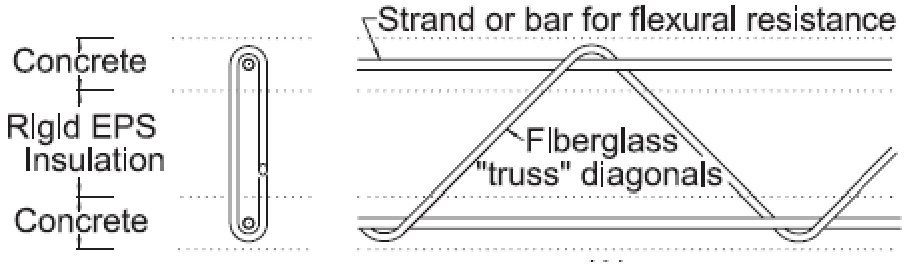

Fig.1: NU-Tie-V1; the early shape of the NU-Tie GFRP connector

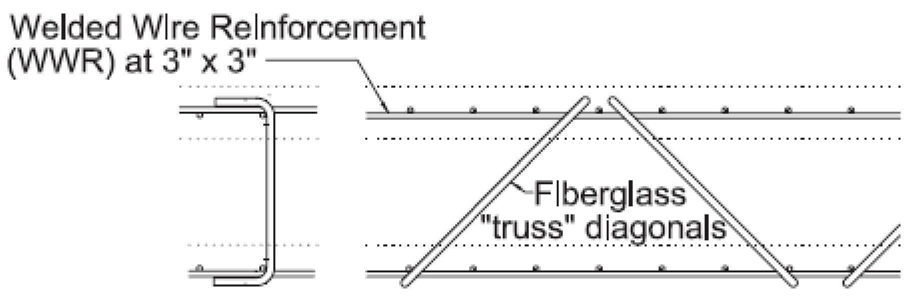

Fig.2: NU-Tie-V2; C-Shaped single GFRP connector

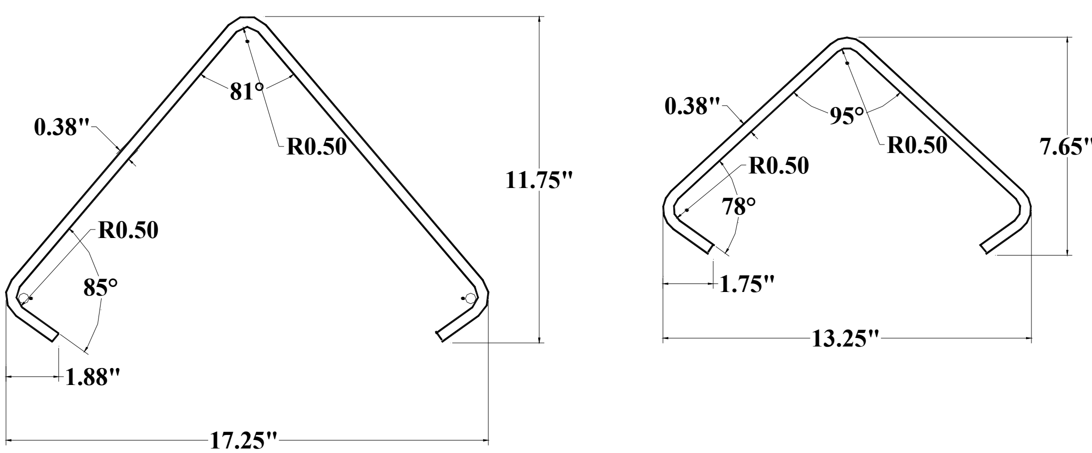

Fig. 3: Details of NU-Tie-V3 (VSS) FRP connectors

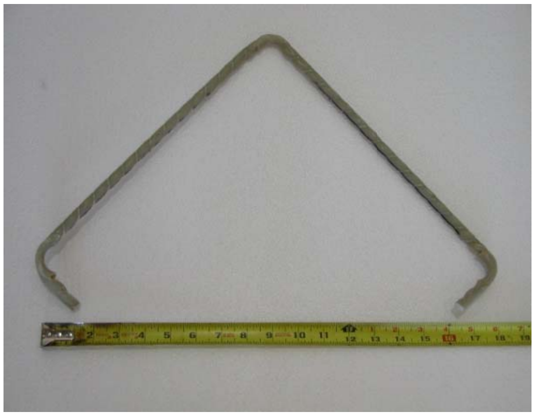

Fig. 4: NU-Tie-V3 (VSS) FRP connector 3-6-3

Fig. 5: NU-Tie-V3 (VSS) FRP connector 2.5-3-2.5

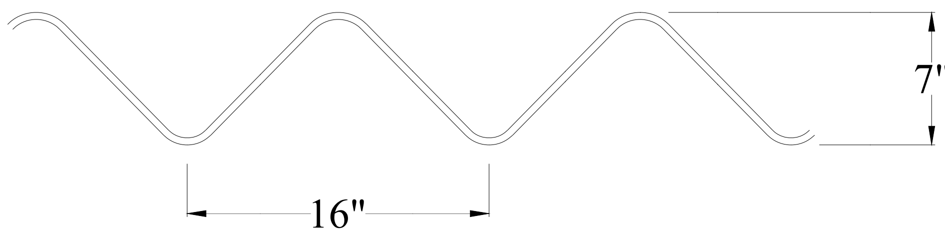

Fig.6: Details of NU-Tie-V4 GFRP connector used for specimens latest specimens with 3in. insulation

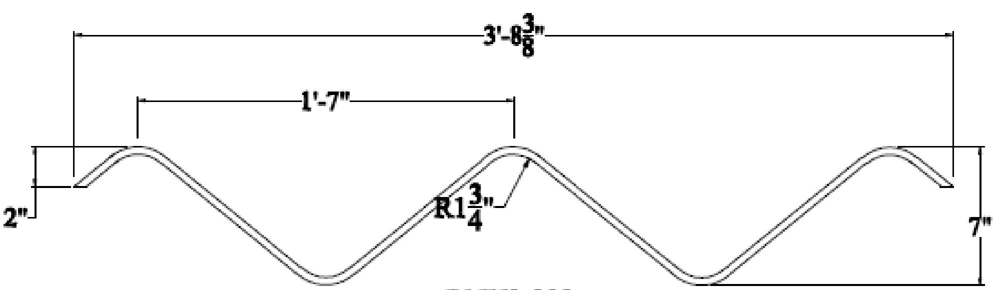

Fig. 7: Details of NU-Tie-V5 GFRP connector for 3in. thick insulation

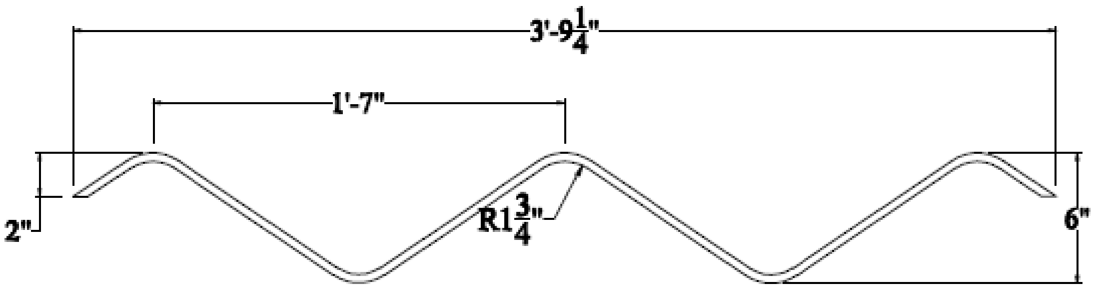

Fig. 8: Details of NU-Tie-V5 GFRP connector for 2in. thick insulation

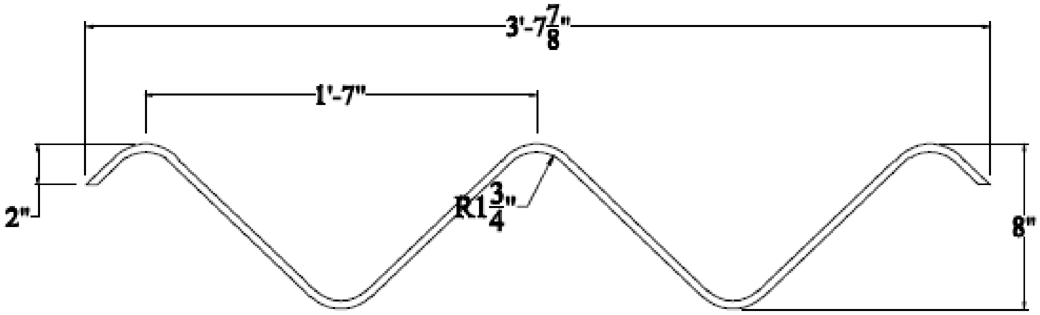

Fig. 9: Details of NU-Tie-V5 GFRP connector for 4in. thick insulation

EXPERIMENTAL PROGRAM

Testing Program

The first phase of the testing program, on NU-Tie-V3 consisted of push-off and full scale flexural tests. The push-off testing was designed to investigate the shear transfer capacity of the NU-Tie and to compare it against other commercially available products. The first one was the lattice fiberglass polymer connector, another patented connector marketed as Delta Tie (DT-C). The second one is the well established welded wire girder, also commercially known as Medco Girder (MC-C). The full-scale testing was designed to investigate the flexural composite behavior provided by the three types of connectors. This testing was intended to verify that the connectors are adequately anchored into the concrete even with no transverse bars placed inside the loops of the NU-Tie. Earlier, it had been believed to be necessary to place these bars, which added complication to the concrete construction process.

The second phase of testing was performed on specimens with NU-Tie-V4. The specimens were cut from an actual prestressed concrete production panel made by Concrete Industries to represent actual products as closely as possible. Another purpose is to investigate the effect of various transverse reinforcement types and amounts.

Push-off Specimen Details

The size of each push-off specimen was 2 by 4 ft. It comprised three concrete wythes and two insulation wythes. To compare shear transfer capacity, the three types of connectors, NU-Tie, DT-C, and MC-C, were used. Thirty specimens were produced. Six specimens were 21 in. thick, and twenty four specimens were 13.5 in. thick. Table 1 shows the details of the specimens. The specimen legend is as follows: NU represents the NU-Tie. R represents residential applications where the concrete wythe thickness is 3” and the insulation thickness is 6” for a total wall thickness of 12”. The letter C represents commercial applications where the concrete wythe is 2.5” thick and the insulation wythe is 3” thick. These dimensions were believed to represent a wide range of possible applications. The symbol CX represents specimens with transverse cross bars through the loops of the NU-Tie. Figures 10-12 show the specimen details.

Table1. Push-off specimen dimensions

	Serial No				
	NU-R	NU-C	NU-CX	DT-C	MC-C
Concrete wythe thickness (in.)	3.0	2.5	2.5	2.5	2.5
Insulation thickness (in.)	6.0	3.0	3.0	3.0	3.0
Total thickness (in.)	21.0	13.5	13.5	13.5	13.5
No. of connectors	4	4	4	4	2
Type of connectors	FRP	FRP	FRP	FRP	WW
No. of specimens	6	6	6	6	6

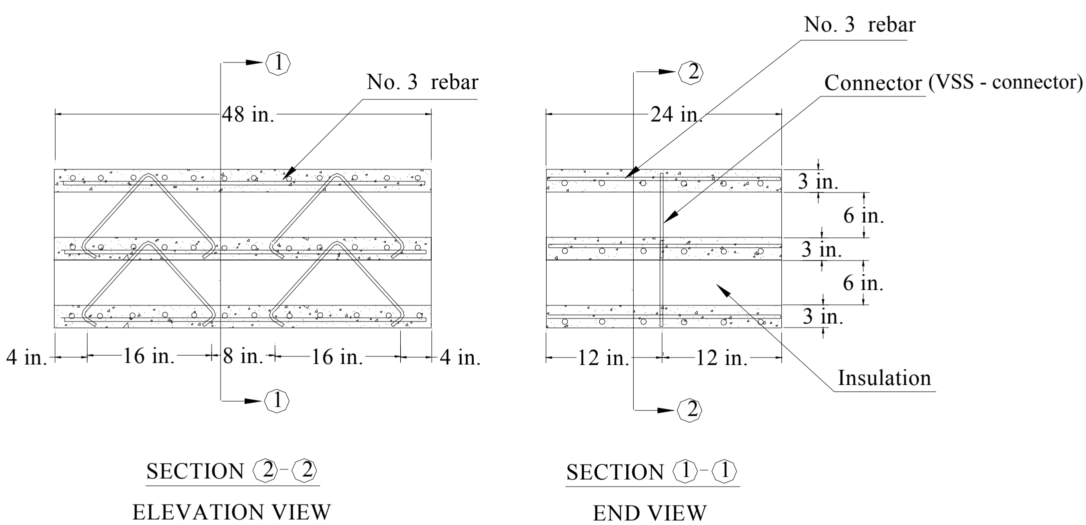

Fig. 10: Details of 2 ft. by 4 ft. by 21-inch push-off specimens, (VSS) FRP connectors

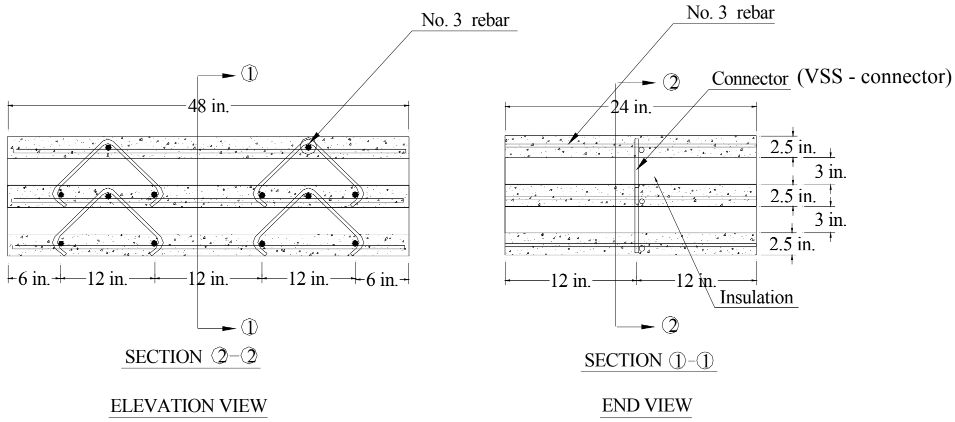

Fig. 11: Details of 2ft. by 4 ft. by 13.5-inch push-off specimens, NU-Tie-V3 connectors

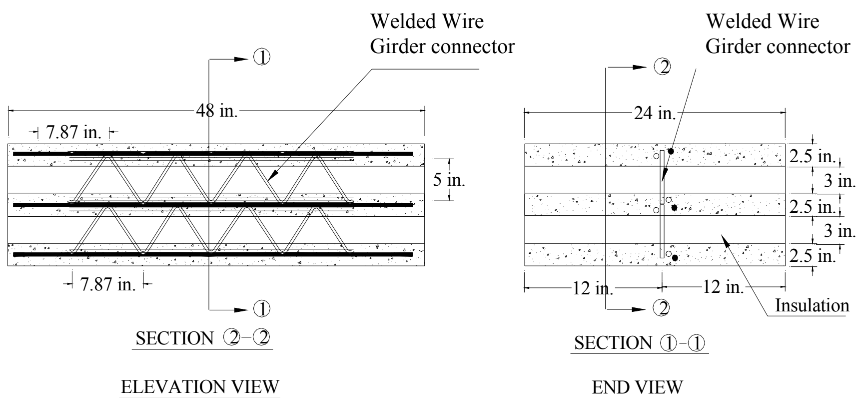

Fig. 12: Details of 2 ft. by 4 ft. by 13.5-inch push-off specimens, welded wire girder connectors

The other two commercial types of composite action connectors considered in this study are the lattice fiber glass polymer tie (DT-C) (Fig. 13), the welded wire girder (MC-C) (Fig. 14).

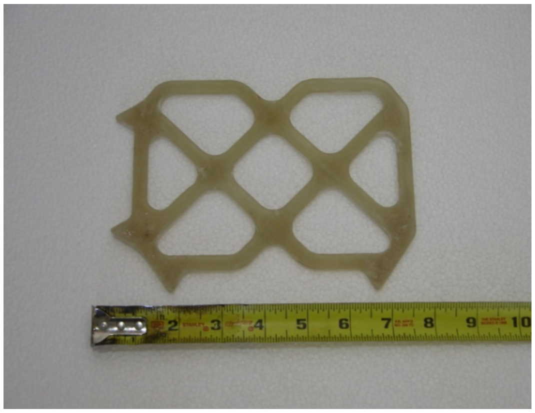

Fig. 13: Lattice Fiber Glass Polymer tie connector

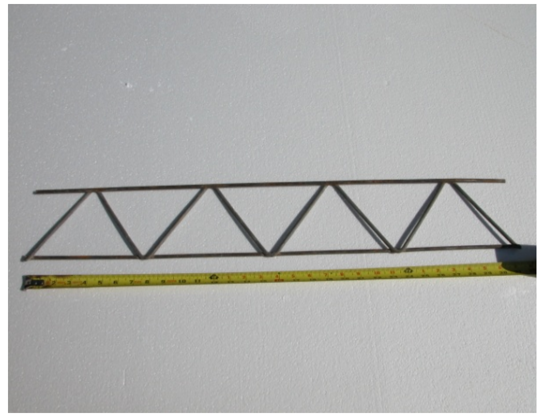

Fig. 14: Welded Wire Girder connector

Flexural Specimen Details

Six specimens, using NU-Tie-V3, connectors were produced for testing that represents wind or earthquake loading on a wall, i.e. transverse loading that produces shear and flexure. Each specimen was 12 in. thick representing the residential reinforced concrete wall as tested in the push-off tests. A representation of the commercial application was done in the second phase of this testing program, using a prestressed concrete panel produced by Concrete Industries.

Each concrete wythe contained a No.3 bar, Grade 60 at 4 inch spacing in both the longitudinal and transverse directions. This was a close representation of the D10xD10@4"x4" reinforcement used on the precast concrete house built in Omaha (see Ref. 11). No prestressing was provided. The reinforcement was adequate to use the wall as a foundation (basement) wall resisting soil pressure up to 10 feet in depth.

Table 2 and Figure 15 give the details of the flexural specimens. Each series of specimens consists of two specimens. The NU-Ties were placed in rows spaced at 2 ft. Two rows were placed in each specimen. For series II, the only change was to double the amount of connectors in each row. Series I and two were cast in a flat position. Series III panels were identical to series I except that the concrete was poured in vertical panel, representing cast-in-place application.

Table 2. Full-scale specimen dimensions

	Series No		
Specimen size 4 by 10 ft	I	II	III
Concrete wythe thickness (in.)	3.0	3.0	3.0
Insulation thickness (in.)	6.0	6.0	6.0
Total thickness (in.)	12.0	12.0	12.0
No. of connectors (NU-Tie-V3)	10	20	10
No. of specimens	2	2	2
Construction method	Horizontally	Horizontally	Vertically

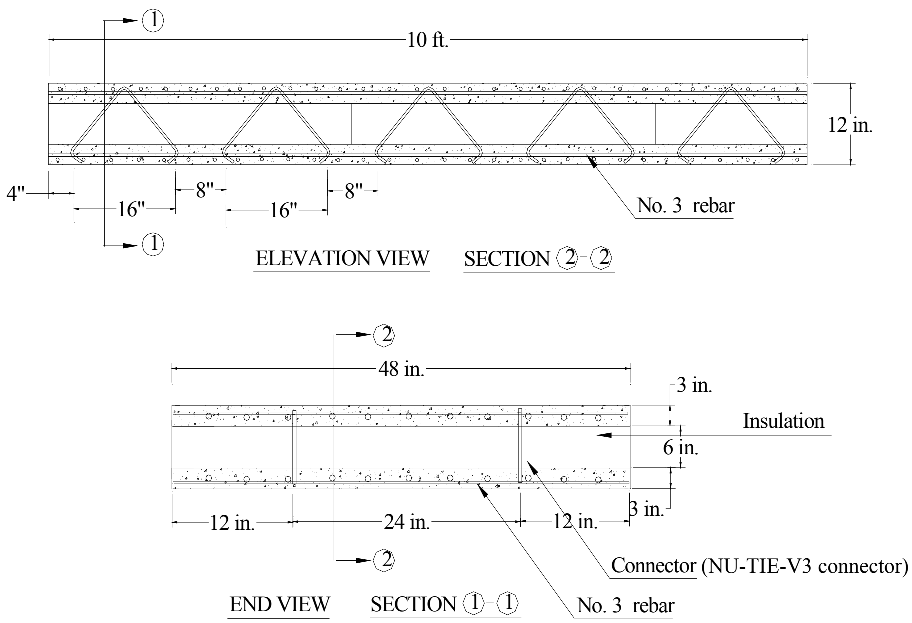

Fig. 15: Details of full-scale specimen (using NU-Tie-V3 connectors)

Phase II- Flexural Testing with NU-Tie-V4

A typical warehouse panel, 32ft tall by 8ft wide, was constructed as a sandwich panel consisting of two prestressed concrete 3 in. thick wythes, with a 3in. thick insulation wythe. The total wall thickness was 9 in. The panel was later saw-cut into eight, 4 ft by 8 ft, specimens. The prestress in an eight foot width consisted of 8- 7/16 in. strands, 4 strands in each wythe. Four sets of two specimens each were tested. They were identical except for the transverse reinforcement: #3 bars at 16 inch spacing, #3 bars at 32 inch spacing, W4 x W4 @ 4in. x 4in., and one set with zero transverse reinforcement. It should be noted that the W4 x W4 @ 4in.x4in. welded wire reinforcement interfered with the strand and NU-Tie positions; it would not be recommended for future use.

All specimens were horizontally constructed, as is the common practice, in the precast concrete plant and shipped to the UNL Structures Laboratory in Omaha. The NU-Tie configuration was the same for all specimens. The ties were place in one line at mid-width of each specimen, i.e. at 2 ft from the edges. Each tie line consisted of 10 legs per specimen, see Table 3 and Fig. 16-18.

Table 3- Flexural Specimen Details for Phase II Testing, Commercial Wall Configuration

	Transversal Reinforcement			
Specimen size 4 by 8 ft	None	#3 @ 16 in.	#3 @ 32 in.	W4 x W4 @ 4in. x 4in.
Concrete wythe thickness (in.)	3.0	3.0	3.0	3.0
Insulation thickness (in.)	3.0	3.0	3.0	3.0
Total thickness (in.)	9.0	9.0	9.0	9.0
No. of NU-Tie –V4 connectors (legs)	10	10	10	10
No. of specimens	2	2	2	2
Longitudinal Pre-stressing / Wythe	2 – 7/16 in. strands	2 – 7/16 in. strands	2 – 7/16 in. strands	2 – 7/16 in. strands
Construction method	Horizontally	Horizontally	Horizontally	Horizontally

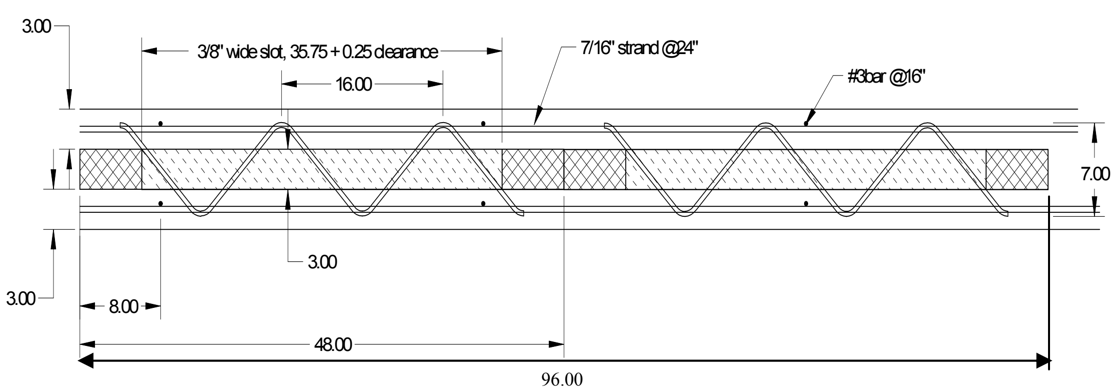

Fig. 16: Details of full-scale specimen (using NU-Tie-V4 connector), with #3 bar @ 16 in. as transversal reinforcement

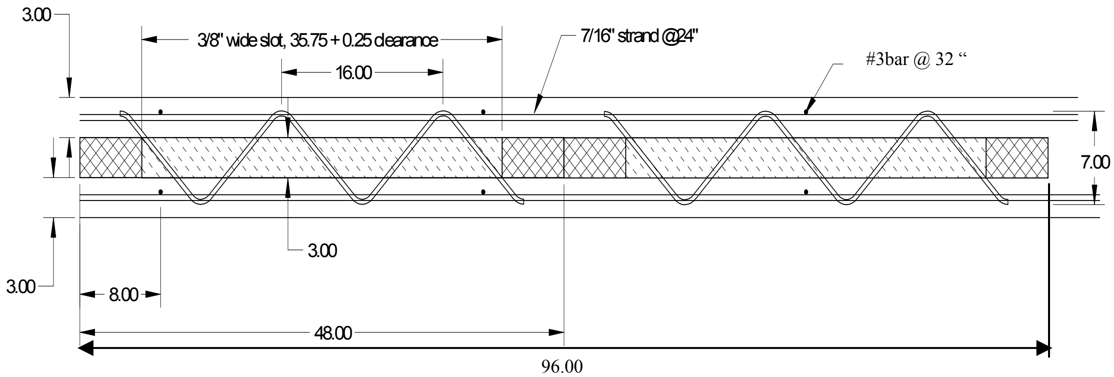

Fig. 17: Details of full-scale specimen (using NU-Tie-V4 connector), with #3 bar @ 32 in. as transversal reinforcement

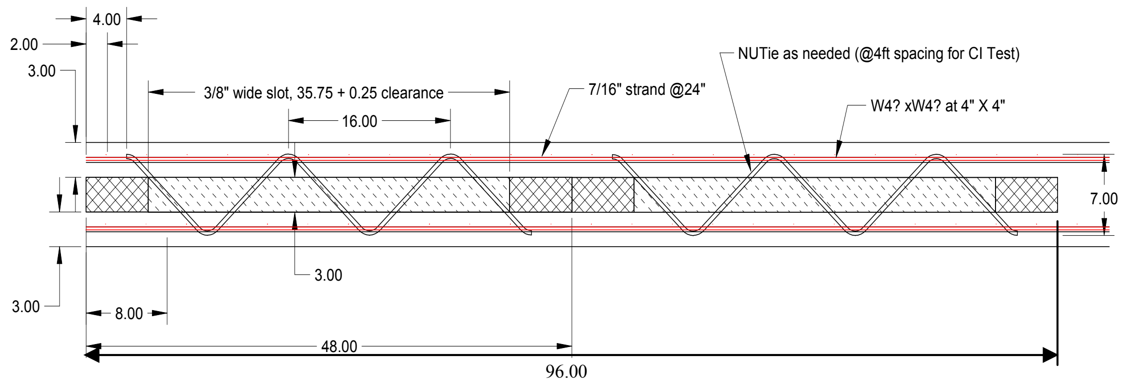

Fig. 18: Details of full-scale specimen (using NU-Tie-V4 connector), with W4 x W4 @ 4in. x 4in. as transversal reinforcement

TESTING SETUP

A test frame was installed for the push-off specimens in the structural laboratory at the University of Nebraska, Omaha. The test frame was anchored to a 500 kip tie-down accessory on the structural testing floor by two high-strength steel threaded rods, see Fig. 19. The specimen was placed vertically in the test frame. Steel tubes, 4 by 4 in., were placed underneath the outer concrete wythes and another steel tube was placed on the middle concrete wythe. Rubber pads were inserted between the steel tubes and concrete wythes. The specimens were tested using a 400 kip hydraulic jack. The attached load cell was used to measure the force applied to the specimens. The specimens were subjected to a concentrically vertical load applied at the middle concrete wythe, which was incrementally increased until failure occurred. The specimens failed when the applied load was reduced from its peak point.

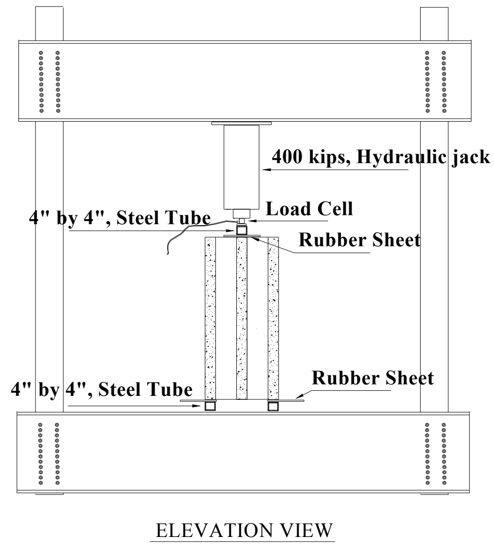

Fig. 19: Push-off test set-up

The test set-up for the flexural specimens is shown in Figs. 20-22. Each specimen was placed horizontally in the test frame. A potentiometer was attached, for deflection measurement, at the center of the span. The specimens were tested by using a 400 kip hydraulic jack placed at the center of the panel. The attached load cell was used to measure the force applied to the specimens. The specimens were then subjected to a vertical load. The load was incrementally increased until failure occurred. Failure of the specimens was regarded when the applied load was reduced from its peak point. The specimens were inspected and observed at specified intervals during testing.

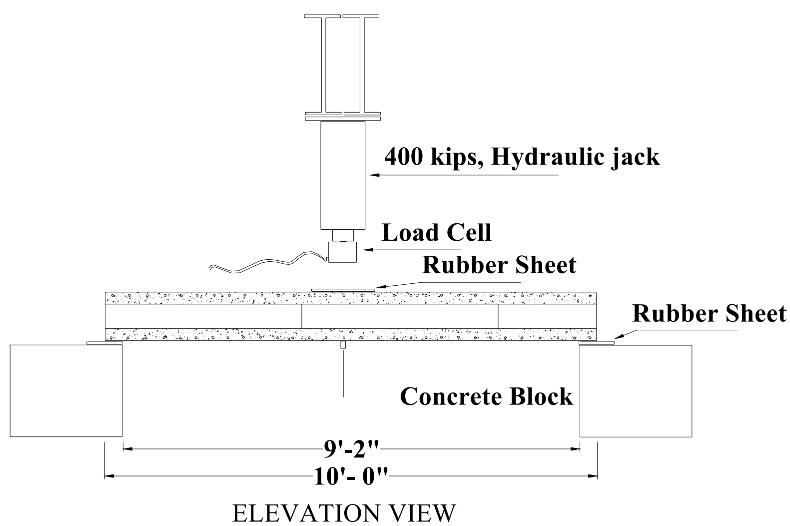

Fig. 20: Full-scale flexure test set-up (for specimens with NU-Tie-V3 connectors)

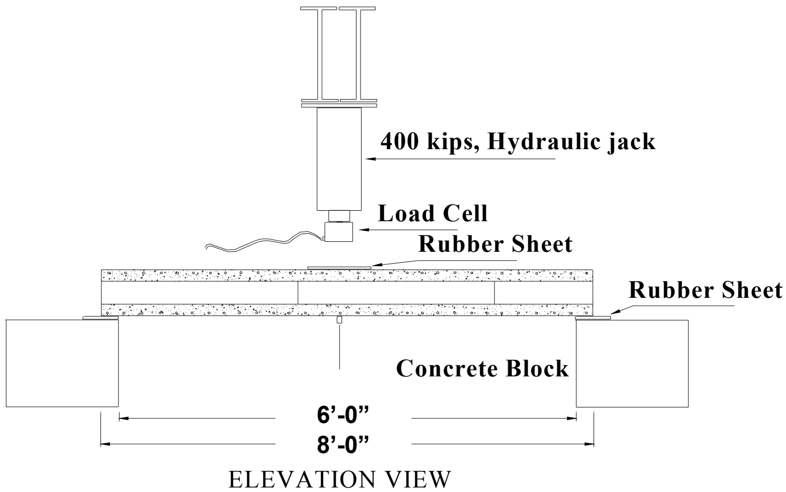

Fig. 21: Full-scale flexure test set-up (for specimens with NU-Tie-V4 connectors)

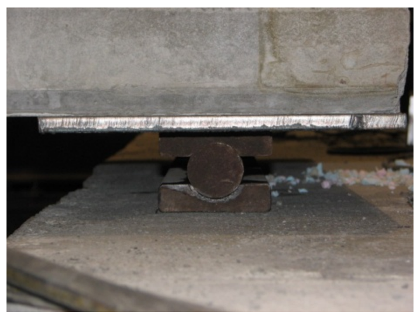

Fig. 22: Full-scale flexure test bearing details (for specimens with NU-Tie-V4 connectors)

COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY

The compressive strength was obtained from compressive tests of concrete cylinders at the same age as their corresponding specimens. The modulus of elasticity was computed as $E_c = 57{,}000\sqrt{f'_c}$ .Tables 4 and 5 show these two concrete properties.

Table 4. Final compressive strength and modulus of elasticity of push-off specimens

Specimen	NU-R		NU-C		NU-CX		MC-C		DT-C	
Concrete wythes	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)
Layer 1	5954	4398	5954	4398	5952	4398	5954	4398	4890	3986
Layer 2	9052	5423	9052	5423	5662	4289	9052	5423	5952	4398
Layer 3	8409	5227	8409	5227	5662	4289	8409	5227	5952	4398

Table 5. Final compressive strength and modulus of elasticity of full-size specimens

Specimen	NU-R-I		NU-R-II		NU-R-V	
Concrete wythes	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)
Layer 1	4890	3986	6063	4438	6531	4606
Layer 2	5662	4289	6063	4438	6531	4606

TENSILE TESTING OF GFRP RODS

Tensile testing of GFRP rods was performed according to the Guide Test Methods for Fiber Reinforced Polymers (FRP) For Reinforcing or Strengthening Concrete Structures prepared by ACI Subcommittee 440.

A 48-in. long GFRP rod was attached to the anchorage device to facilitate the gripping dimension, as shown in Fig. 23. An Extensometer, a strain measuring device, was mounted at the middle of the specimen, as shown in Fig. 24. Six specimens were tested under a loading rate of 6000 lbs/min. The test results are shown in Table 6. Tensile strength, ultimate strain and modulus of elasticity were calculated as follows:

Tensile strength $f_u = \dfrac{F_u}{A}$

where f_u = *tensile strength (psi)*

F_u = *load failure (lbs)*

A = *average cross section area (in^2)*

Tensile modulus of elasticity $E_L = \dfrac{F_1 - F_2}{(\varepsilon_1 - \varepsilon_2)A}$

where EL = *axial modulus of elasticity (psi)*

A = *cross section area* (in^2)

F_1 *and* ε_1 = *load and corresponding strain, respectively, at approximately 50% of the ultimate tensile capacity or guaranteed tensile capacity*

F_2 *and* ε_2 = *load and corresponding strain, respectively, at approximately 20% of the ultimate tensile capacity or guaranteed tensile capacity*

Ultimate strain $$\varepsilon_u = \frac{F_u}{E_L A}$$

where ε_u = *ultimate strain of FRP bar*

Ductility is the ratio of the strain at failure to the strain at yielding. Fig. 25 shows the stress-strain relationship obtained from the tensile test. Stress and strain increased in a linear proportion to failure with no yield point. It can be considered that GFRP reinforcement is a non-ductile material.

The test results may be different since the tensile strength and modulus of elasticity of FRP depend on the fiber content and manufacturing process, as well as quality control.

Table 6. Tensile test results

Sample No	Average Gross section area (in2)	Load Failure (lbs)	Tensile Strength (psi)	Ultimate Strain (in/in)	Modulus of Elasticity (psi)
1	0.11	15610	141909	0.0222	6.736×10^6
2	0.11	15,440	140364	0.0145	6.202×10^6
3	0.11	15500	140909	0.0142	6.514×10^6
4	0.11	15310	139182	0.0284	6.628×10^6
5	0.11	15950	145000	0.0466	6.483×10^6
6	0.11	15390	139909	0.0194	6.726×10^6
Ave.		15533	141212	0.0242	6.548×10^6

Fig. 23: Facilitate gripping

Fig. 24: Extensometer

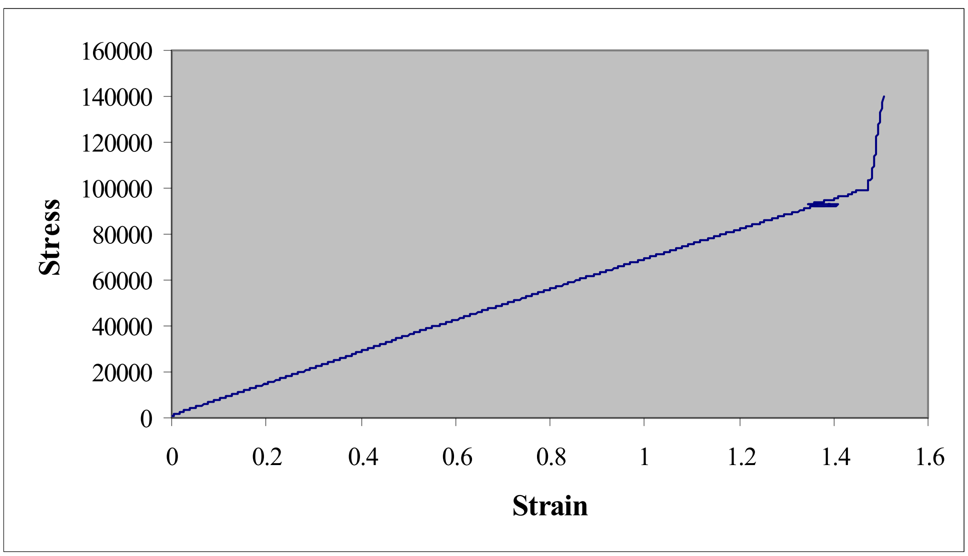

Fig. 25: FRP stress-strain relationship

Tensile testing of GFRP rods was performed according to the Guide Test Methods for Fiber Reinforced Polymers (FRP) For Reinforcing or Strengthening Concrete Structures prepared by ACI Subcommittee 440. The test results provided by Hughes Bros. showed that average tensile strength was 122,699 (psi), and the average ultimate strain was 0.0206 (in./in.). The average modulus of elasticity was found to be 5,980,390 (psi). Phase I results showed the ability of these bars to have a tensile strength of 140 ksi and higher. However, due to variability of material and the records provided by Hughes Bros. it is recommended that the strength of the GFRP bars be specified as 120 ksi.

TEST RESULTS

The failure behavior of specimens serial NU-R, NU-C, NU-CX was very similar. Failure of GFRP connector was due to shear and flexure, as seen in Figs. 26 (a) and (b). When loading was in process, the applied load was abruptly dropped and then increased after each connector failure until ultimate load was reached. The test results of the push-off specimens are summarized in terms of ultimate load in Table 7. The relationship between ultimate load and total area of connector (P_u/A_s) was computed as the Ultimate load /Total area (wire or GFRP).

(a) (b)

Fig. 26 (a), (b): Failure of NU-Tie-V3 GFRP connectors

Table 7. Push-off test results (for specimens with NU-Tie-V3 connectors)

Specimen					Connector		Test results	
Serial No.	No.	Thickness		f'c (psi)	Type	Area (in^2)	Ultimate Load (lbs.)	Pu/As (psi.)
		Con.	Ins.					
NU-R	1	3	6	5954	GFRP	0.88	25,417	28,883
	2	3	6	5954	GFRP	0.88	26,233	29,810
	3	3	6	5954	GFRP	0.88	24,910	28,307
	4	3	6	5954	GFRP	0.88	33,923	38,549
	5	3	6	5954	GFRP	0.88	25,130	28,557
	6	3	6	5954	GFRP	0.88	30,613	34,788
							27,704	**31,482**
NU-C	1	2.5	3	5954	GFRP	0.88	37,148	42,214
	2	2.5	3	5954	GFRP	0.88	31,790	36,125
	3	2.5	3	5954	GFRP	0.88	40,191	45,672
	4	2.5	3	5954	GFRP	0.88	36,524	41,505
	5	2.5	3	5954	GFRP	0.88	28,306	32,166
	6	2.5	3	5954	GFRP	0.88	33,201	37,728
							34,526	**39,235**
NU-CX	1	2.5	3	5662	GFRP	0.88	27,798	31,589
	2	2.5	3	5662	GFRP	0.88	30,709	34,897
	3	2.5	3	5662	GFRP	0.88	36,405	41,369
	4	2.5	3	5662	GFRP	0.88	34,406	39,098
	5	2.5	3	5662	GFRP	0.88	37,058	42,111
	6	2.5	3	5662	GFRP	0.88	35,413	40,242
							33,631	**38,218**
DT-C	1	2.5	3	4890	P	1.25	25,123	20,098
	2	2.5	3	4890	P	1.25	26,196	20,957
	3	2.5	3	4890	P	1.25	28,257	22,606
	4	2.5	3	4890	P	1.25	22,727	18,182
	5	2.5	3	4890	P	1.25	23,932	19,146
	6	2.5	3	4890	P	1.25	24,079	19,263
							25,052	**20,040**
MC-C	1	2.5	3	5954	WW	0.64	37,089	57,952
	2	2.5	3	5954	WW	0.64	37,971	59,330
	3	2.5	3	5954	WW	0.64	39,669	61,983
	4	2.5	3	5954	WW	0.64	40,410	63,141
	5	2.5	3	5954	WW	0.64	38,081	59,502
	6	2.5	3	5954	WW	0.64	35,803	55,942
							38,170	**59,641**

Area = Total area of diagonal welded wire or GFRP

NU Area of GFRP = (2x4x0.11) = 0.88 in^2 (0.375 in Dia.)

MC Area of Welded wire = (2x8x0.04) = 0.64 in^2 (0.225 in Dia.)

DT Area of GFRP = (4x2.5x1.25) = 1.25 in^2 (0.125 in Thickness)

The failure behavior of flexural specimens of Phase I was very similar. Visual cracks formed at the bottom fiber when the applied load was in the range of 12-15 kip. Horizontal shear failure occurred when the outermost connector was brittle. Crushing of concrete due to flexure failure occurred in the top concrete wythe when the second row of connectors failed. Flexure failure took place in the bottom concrete wythe when the ultimate load was reached. Observations revealed that loading did not continuously increase. Similar to the push-off test, the applied load abruptly dropped and then increased at every single connector failure until ultimate load was reached. Ultimate load and deflection results are given in Table 8.

Table 8. Full-scale flexure test results (for specimens with NU-Tie-V3connectors)

Specimen					Connector		Test results	
Serial No.	No.	Thickness		f'c (psi)	Type	Area (in^2)	Ultimate Load (lbs.)	Deflection (in)
		Con.	Ins.					
I	1	3	6	4890	FRP	1.1	19,399	2.81
	2	3	6	4890	FRP	1.1	20,931	2.28
II	3	3	6	6063	FRP	2.2	23,891	2.55
	4	3	6	6063	FRP	2.2	20,681	2.95
III	5	3	6	6531	FRP	1.1	16,095	3.12
	6	3	6	6531	FRP	1.1	20,956	2.00
Ave.							20,325	2.13

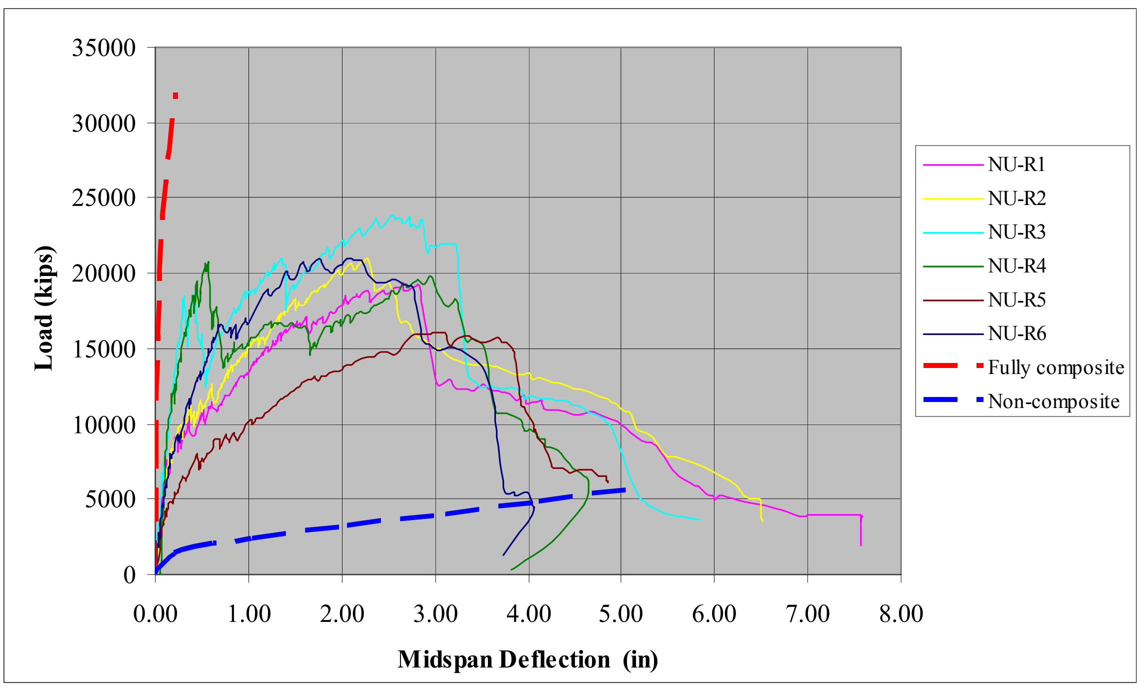

Fig. 27: Mid-span deflection (for specimens using NU-Tie-V3 connectors)

The load-deflection curves for the specimens of Phase II is shown in Fig. 28. The figure also shows the theoretical load-deflection relationships for a non-composite section and for a fully composite member for comparison purposes. Similar to the push-off test, the applied load abruptly dropped and then increased at every single connector failure until ultimate load was reached. The Phase II test results are given in Table 9.

Table 9. Full-scale flexure test results (for specimens using NU-Tie-V4 connectors)

Specimen					Connector		Test results	
Transverse Rein.	No.	Thickness Con.	Thickness Ins.	f_c' (psi)	Type	Area (in^2)	Ultimate External Load (lbs.)	Deflection at Ultimate Load (in)
No-Rein.	1	3	3	5000	GFRP	1.1	16,847	0.63
	2	3	3	5000	GFRP	1.1	18,150	1.12
#3 @ 32”	3	3	3	5000	GFRP	1.1	19,657	0.92
	4	3	3	5000	GFRP	1.1	16,644	0.87
#3 @ 16”	5	3	3	5000	GFRP	1.1	17,496	0.72
	6	3	3	5000	GFRP	1.1	14,486	0.42
W4xW4 @ 4”x4”	7	3	3	5000	GFRP	1.1	16,826	1.96
	8	3	3	5000	GFRP	1.1	21,448	1.67
Ave.							17,694	1.04

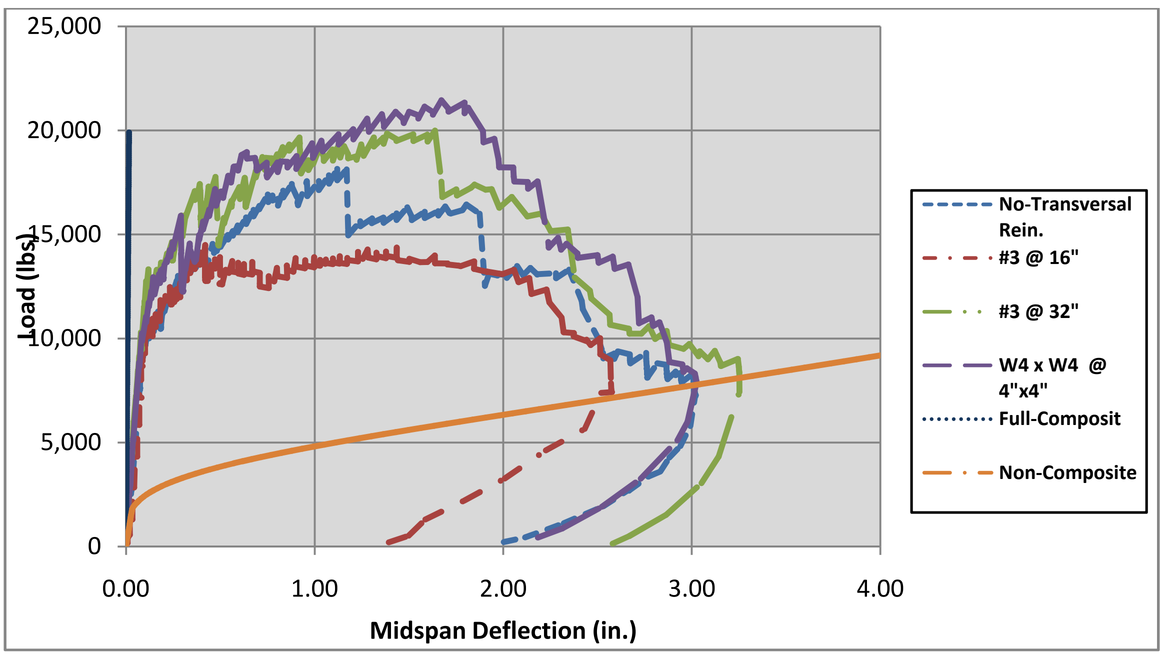

Fig. 28: Mid-span deflection (for specimens using NU-Tie-V4 connectors)

DISCUSSION

Deflections

The load and mid-span deflection of the six specimens is shown in Fig. 27. The figure also shows the theoretical predictions which were calculated based on the assumption of fully composite and non-composite behavior. It shows that the actual deflection is larger than that achieved by theory using full composite action. The reasons include the flexibility of the connectors, compared to solid concrete connection. In addition, the GFRP bars have lower modulus of elasticity as noted above.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad I_e \le I_g \tag{1}$$

$$M_{cr} = \frac{f_r I_g}{y_t} \tag{2}$$

$$f_r = 7.5\sqrt{f_c'} \tag{3}$$

$$\Delta_i = \frac{5 M_a L^2}{48 E_c I_e} \tag{4}$$

Development of GFRP reinforcement

The test results show that no direct pull-out failure occurred. The average ultimate load of specimen series NU-C and NU-CX was 34,526 and 33,631 respectively, which indicated little benefit from threading a bar through the loop of the NU-Tie. The ties were embedded 1.5" in some of the tests and 2" in others. It is recommended that the embedment be 2" with a tolerance of plus or minus ½". With this recommendation, the NU-Tie would be expected to have adequate anchorage into the concrete without any need for cross bar anchors. Based on the testing it will be shown that #3 @ 32 In. transverse reinforcement is the minimum amount required for this system. But these bars do not have to be inserted into the loops of the NU-Tie. They can be placed adjacent to the ties and would still contribute to anchorage as shown in Phase II testing.

Shear friction

$$V_n = A_{vf} f_y \left(\mu \sin\alpha_f + \cos\alpha_f\right) \tag{5}$$

Horizontal shear strength

$$V_{nh} = \left(260 + 0.6\,\rho_v f_y\right)\lambda b_v d \quad < \quad 500 b_v d \tag{6}$$

Composite behavior

The ultimate load obtained from the test results is lower than the design load, based on the assumption of fully composite behavior, because the shear transfer strength of the connector was overestimated. As shown in Table 6, the average stress of GFRP reinforcement is 31,482 psi and 38,727 psi for specimens with 6 in. and 3 in. insulation, respectively. This average stress is much lower than that of the average tensile strength of 120-140 ksi. Therefore, the FRP connection did not fail in tension alone but rather in combined flexure-shear-axial load effects.

Consider the sandwich panel subjected to a vertical load, as shown in Fig 29. The applied load would create tension in the bottom fiber and compression in the top fiber, thereby causing slip in the bottom concrete, insulation and top concrete wythe. The separation would create flexure and tensile stress in the connector which, in turn, would create compressive stress in another leg of the connector. The tensile stress, compressive stress and flexure occur at the same time; therefore the lowest stress governs the strength of the connector.

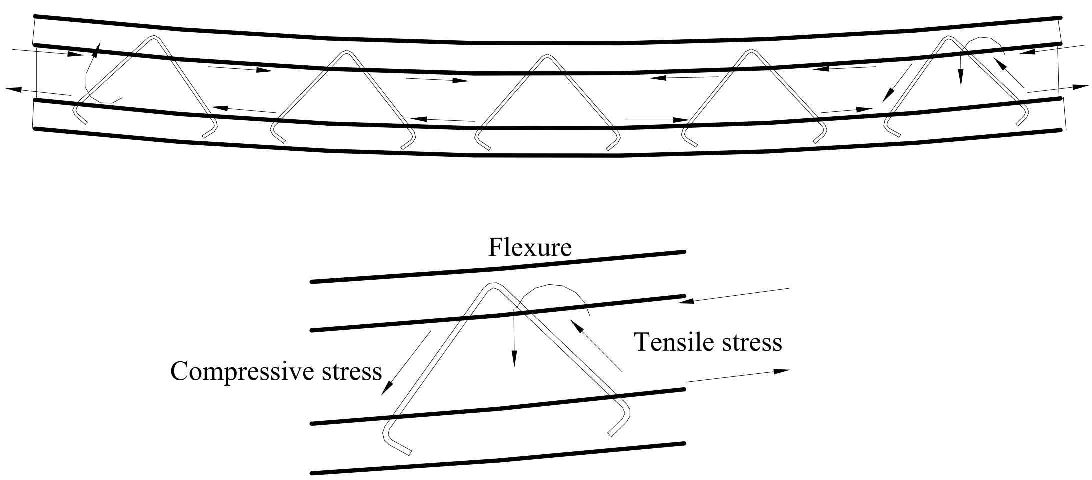

Fig. 29: Forces in the NU-Tie-V3 connector

Due to this complex interaction, it recommended that designers use the lowest values achieved in the testing programs. Please see the example in Appendix A for illustration of how to determine the capacity for design purposes.

Effect of Amount of Transverse Reinforcement

As shown before in Table 9, the amount transverse reinforcement arrangements did not have a significant effect on the panel capacity or behavior. One exception was the case where no reinforcement was provided. In that case, a large longitudinal crack developed at the top surface of the specimen along the tie line, indicating a splitting failure.

In all cases, the specimens displayed a large capacity for resisting lateral load, much larger than would be required from wind loading. To illustrate, the wind loading capacity was calculated for each of Phase II specimens, assuming a 32 ft tall wall. Table 10 shows an equivalent wind loading of over 150 psf, compared to the 25 psf typically assumed for design.

Table 10. Equivalent wind load based on Full-scale flexure test results (for specimens using NU-Tie-V4 connectors)

Specimen				Connector		Test results	Equivalent Wind load for 32ft tall panel simply supported (psf)
Transverse Rein.	Thickness		f'c (psi)	Type	Area (in^2)	Average Ultimate External Load (lbs.)	
	Con.	Ins.					
No-Rein.	3	3	5000	GFRP	1.1	17,498	156.2
#3 @ 32"	3	3	5000	GFRP	1.1	18,151	162.0
#3 @ 16"	3	3	5000	GFRP	1.1	15,991	142.8
W4xW4 @ 4"x4"	3	3	5000	GFRP	1.1	19,137	170.8
Ave.						17,694	158.0

CONCLUSIONS AND RECOMMENDATIONS

The test results show that NU-Ties have adequate strength for design of composite sandwich wall panels. It was found that transverse reinforcement had practically no effect on the capacity or behavior of the panels, as long as a minimal amount of #3 @ 32 in. is provided. Based on this research, it is recommended that the conservative stress limits shown in the example in Appendix A be used in design. The recommended tie capacity accounts for the fact that the tie is not subject only to axial force but to a combined axial and shear forces and bending moments. Designers are recommended to refer to the PCI Design Handbook, Section 9.4, for recommendations on design of precast sandwich panels. The exceptions to that procedure are: The moment of inertia (I) for panels using NU-Tie-V4 or NU-Tie-V5 should be replaced by 80% of the moment of inertia of the fully-composite section, for P-Delta effects. The same value for (I) should be used in calculating thermal bowing effect (PCI Design Handbook section 4.8.5) on the panels using NU-Tie-V4 or NU-Tie-V5. The amount of prestressing strand, or mild reinforcement for non-prestressed members, should be determined on the assumption that the flexural strength of the composite member is 50 percent of that of a solid concrete wall. This 50% ratio is conservative at this time and is subject to refinement with further research. The amount of NU-Tie should be determined based on the stress in the leg located at the maximum shear section, see the example in Appendix A.

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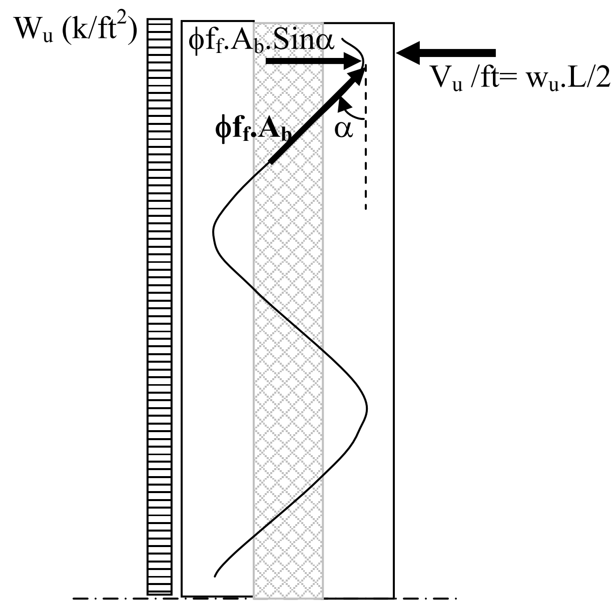
Appendix A- Design Example

Determination of Maximum Spacing for NU-Tie for a Panel Subject to a Uniformely Distributed Load:



Consider a strip of 1 ft width.

V_u (kips/ft width) = W_u.L/2

The component of tie force to resist this shear (per unit width of the panel) is equal to $\phi c_e f_f \frac{A_b}{S} \sin\alpha$

Where,

ϕ, the strength reduction factor taken equal to 0.75

f_f, effective tie capacity = 0.5*f_u = 0.5x120 = 60 ksi.

f_u, breaking strength of #3 tie, taken equal to 120 ksi.

c_e, exposure factor equal to 0.7

A_b, cross sectional area for #3 tie = 0.11 in.^{2}

S, spacing between tie rows (ft).

α, angle of tie leg with the axis of panel, degrees.

For $\alpha = 45^\circ$

$$0.75*0.5*0.7*120*\frac{A_b}{S}*\sin 45^\circ = W_u.L/2 = V_u$$

$$S = 22.274\ \frac{A_b}{V_u}$$

<u>For #3 tie:</u>

$$\boxed{S_{max} = \frac{2.45}{V_u}}$$

Where,

S_{max}, maximum allowed spacing between ties.

V_u, end shear due to ultimate factored load on the panel (k/ft)

To apply these formulas numerically, consider a 32'x10' panel subject to wind load of 25 psf.

W_u = 1.6 x 0.025 = 0.04 ksf

L = 32 ft

V_u /ft width = W_u.L /2

= 0.04x32/2 = 0.64 kips / ft

$$S_{max} = \frac{2.45}{V_u}$$

$$S_{max} = \frac{2.45}{0.64} = 3.8 \text{ ft.}$$

The maximum spacing between #3 NU-Tie rows for the panel under consideration = 3.8 ft.