INSULATED PRECAST CONCRETE PANELS FOR STRUCTURES REQUIRING DRYWALL FINISH

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Abstract

The mechanical properties and thermal resistance of glass fiber-reinforced polymer (GFRP) ties have made them excellent choice as shear connectors in precast concrete sandwich panels (PCSPs) for building components where structural and thermal efficiencies are needed. Numerous research projects have been conducted during the last two decades and resulted in the truss-shaped design of GFRP ties that links the two wythes of concrete through an insulation layer of extruded polystyrene in a simple, production-friendly, and cost effective manner. This paper deals with a new application of the truss-shaped GFRP ties in precast walls when it is a requirement to hide the joint between precast panels behind a drywall finish. The new wall system will be used in residential construction, and commercial applications such as office buildings, schools and churches. The unique design of these panels allows the installation of electrical conduits, behind the drywall sheets without having to predetermine block out locations as in standard sandwich panels. The system still maintains the structural efficiency, thermal and sound insulation, and speed of construction of traditional PCSPs. This paper presents the design and production of two 10 ft wide, 9 in. thick, and 20 ft and 16 ft long specimens using #3 GFRP ties. The main objective of this research is to experimentally verify the flexural capacity, horizontal shear capacity, and deflection of the new panels. The two panels were fabricated and tested at Northeast Precast plant in NJ. Testing results indicated that the design of the panels is adequate for resisting a factored wind pressure of 110 psf and the number, size, and distribution of GFRP ties used in the tested panel are adequate for resisting horizontal shear and achieving the flexural capacity of a fully-composite panel. Panel deflection predicted using truss model was found to be very close to actual panel deflection. The tested panels failed by the pullout of the ties rather than the rupture of the ties, which indicated that a deeper embedment would have resulted in utilizing the full capacity of the GFRP ties.

Keywords: Sandwich Panel, Shear Connector, Composite Action, Flexural Capacity, Drywall, Thermal Insulation.

INTRODUCTION

Precast concrete sandwich panel (PCSP) is a structurally and thermally efficient system that is used for exterior walls in residential and commercial buildings. A typical PCSP consists of two precast concrete wythes separated by a layer of insulation (e.g. Extruded Polystyrene (XPS)) 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 those 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. A special connector made of glass fiber-reinforced polymer (GFRP) ties that have excellent thermal and mechanical properties was developed and patented by researchers at the University of Nebraska-Lincoln (UNL) in August 15, 1995.[2,3]

During the last two decades, numerous research projects were conducted on these GFRP ties that resulted in the generations shown in Figure 1. The current design has evolved from a looped tie stretching in the longitudinal direction (first generation), to plane truss diagonals with various depths and angles to fit different panel thicknesses (fifth generation). The investigation presented in this paper uses the fifth generation of GFRP ties.

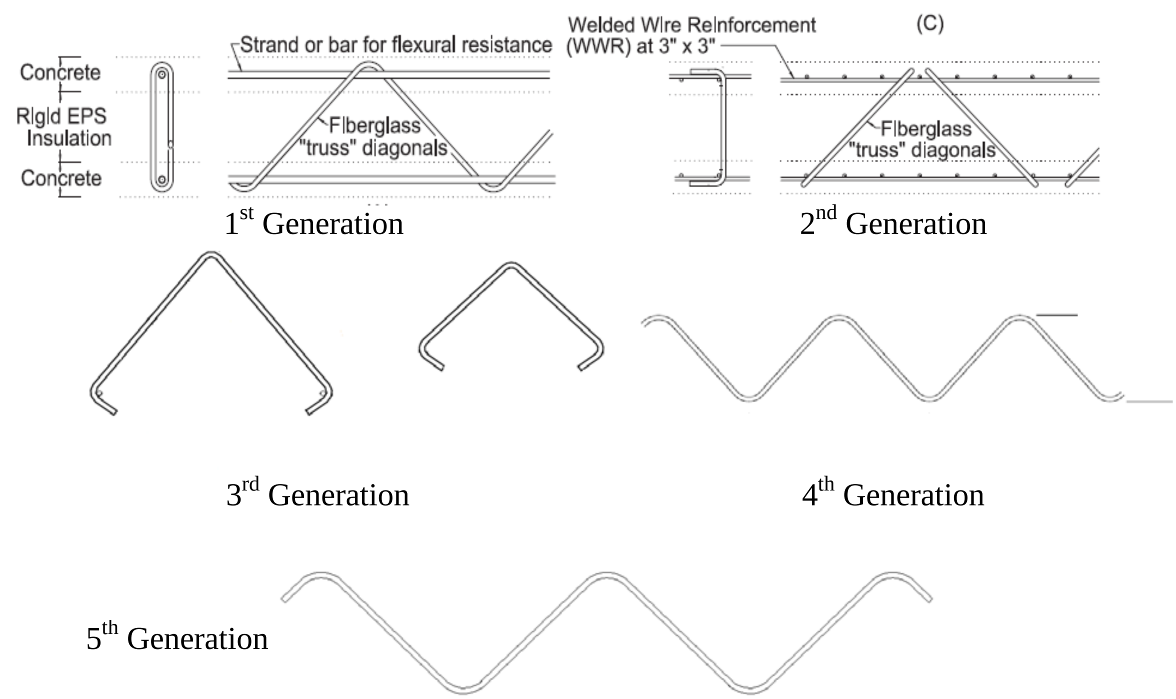


Fig. 1: Generations of GFRP connectors

This paper deals with a new application of the GFRP ties in precast walls when it is a requirement to hide the joint between precast panels behind a drywall finish. The new wall system will be useful for residential construction and commercial applications, such as office buildings, schools and churches. The unique design of these panels, shown in Figure 2, allows the installation of electrical conduits, behind the drywall sheets without having to predetermine block out locations as in standard sandwich panels. The system still maintains the structural efficiency, thermal and sound insulation, and speed of construction of traditional PCSPs[4].

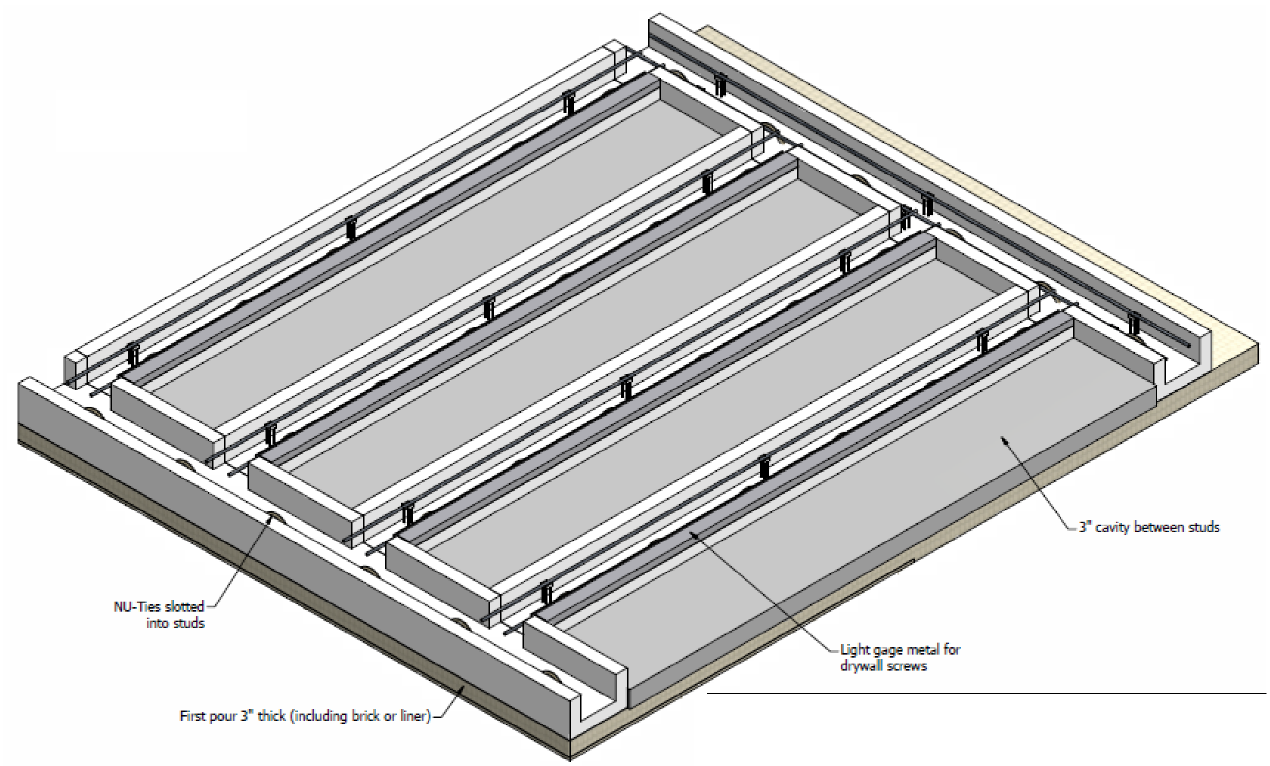


Figure 2: Proposed precast concrete panel for structures requiring drywall finish

The objective of this paper is to experimentally verify the design and production of the new panels. Two 10 ft wide, 9 in. thick, and 16 ft and 20 ft long specimens, shown in Figure 3 and 4 respectively, were fabricated and tested at Northeast Precast plant in NJ. The following section presents the design and production of the two specimens. The third section presents testing procedures and results. The last section summarizes the research conclusions.

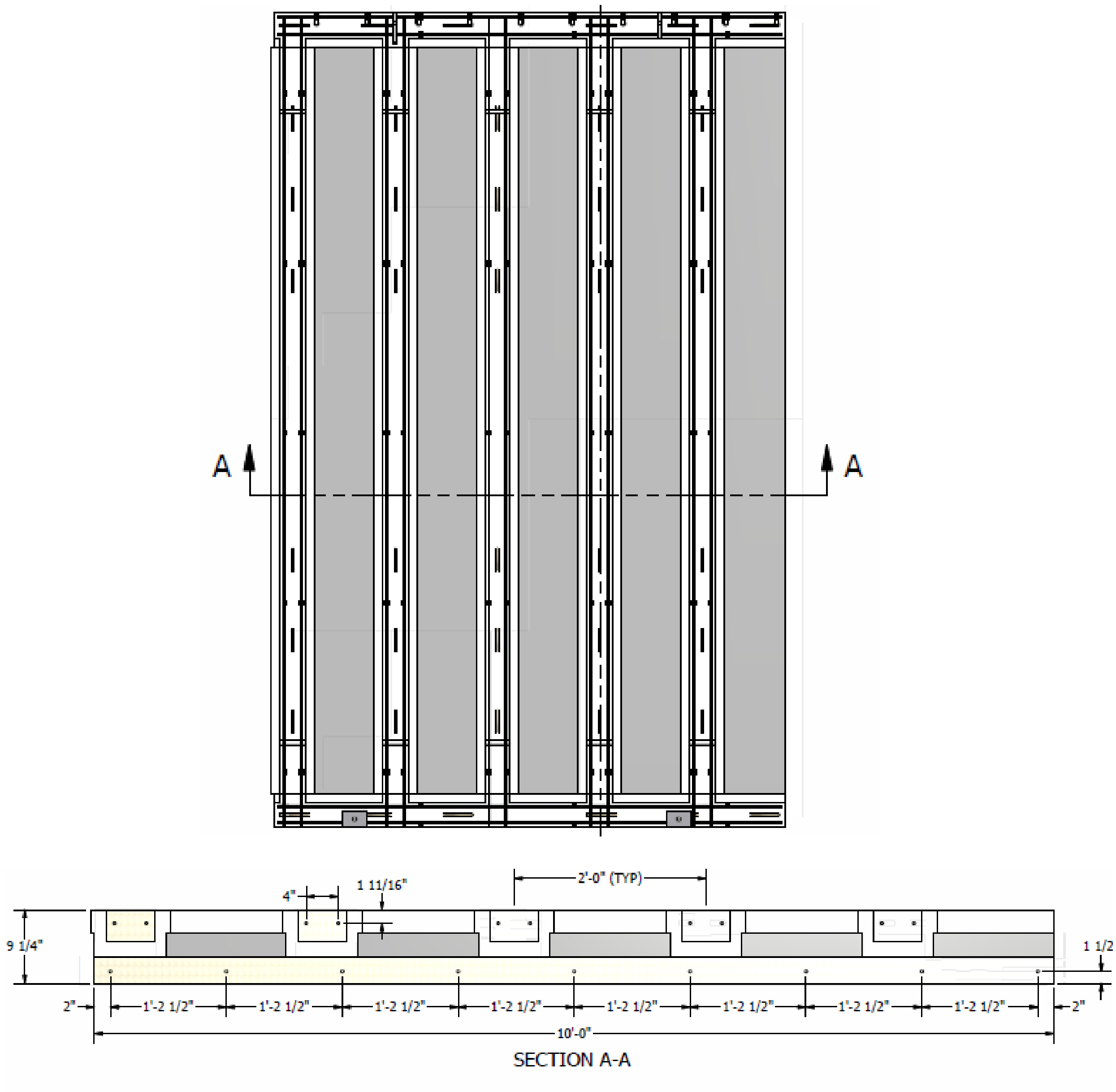


Figure 3: Plan and section A-A of the 16 ft long specimen

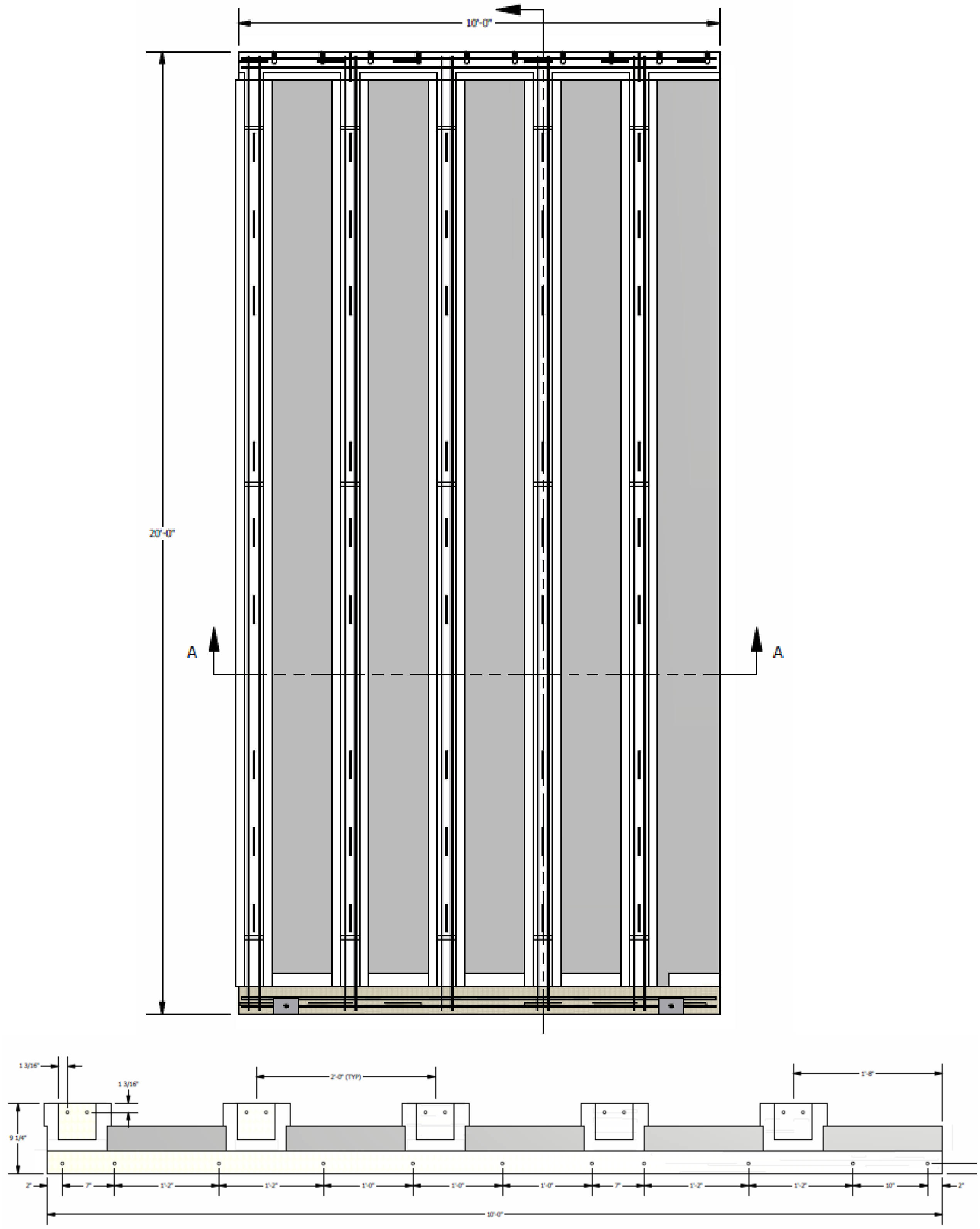
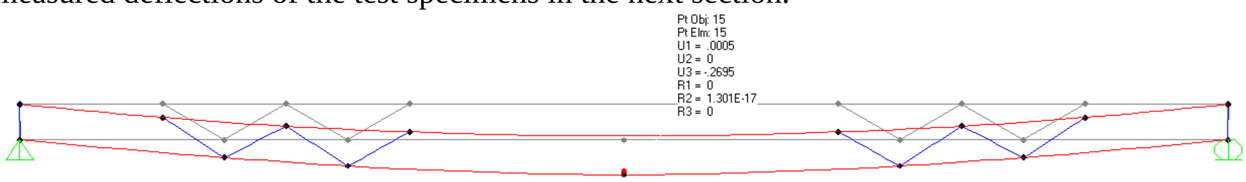


Figure 4: Plan and section A-A of the 20 ft long specimen

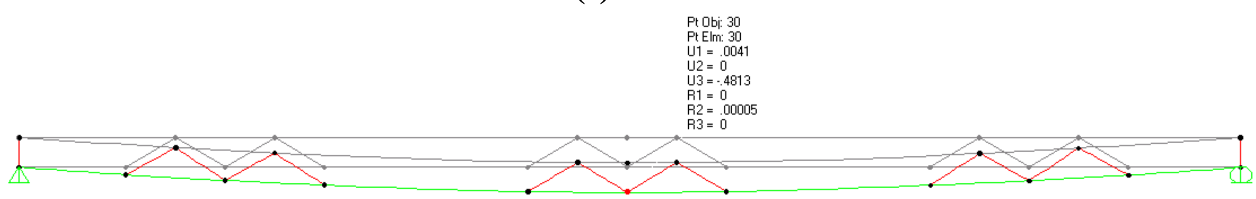
DESIGN AND PRODUCTION

The two test panels shown in Figures 3 and 4 were reinforced using #3 Grade 60 bars and #3 GFRP ties as shear connectors The GFRP ties used in this investigation are 7 in. (178 mm) high and made of 3/8 in. (9.5 mm) diameter GFRP bent rods, which have a cross-sectional area of 0.11 in^2 (71 mm^2). The tensile strength testing of the GFRP ties was performed according to the Guide Test Methods for Fiber Reinforced Polymers for Reinforcing or Strengthening Concrete Structures prepared by ACI Subcommittee 440K.[5] Test results provided by the manufacturer showed an average tensile strength of 122.7 ksi (847 MPa), average ultimate strain of 0.0206 (in./in.), and average modulus of elasticity of 5,980 ksi (41,262 MPa). Although testing performed by the authors had shown a tensile strength of up to 140 ksi, the manufacturer recommends that the specified guaranteed tensile strength and modulus of elasticity of 110 ksi (759 MPa) and 5920 ksi (40,848 MPa), respectively, are used in design.

A truss model was used to predict the panel deflections under its self-weight, which is 50 psf. In this model, the bottom concrete wythe is modeled as the truss bottom chord members, while the concrete studs were modeled as the truss top chord members. The truss diagonal members represent the GFRP ties. Figures 5(a), and (b) shows the truss models of the 16 ft and 20 ft long specimens respectively. The predicted deflections are compared versus the measured deflections of the test specimens in the next section.



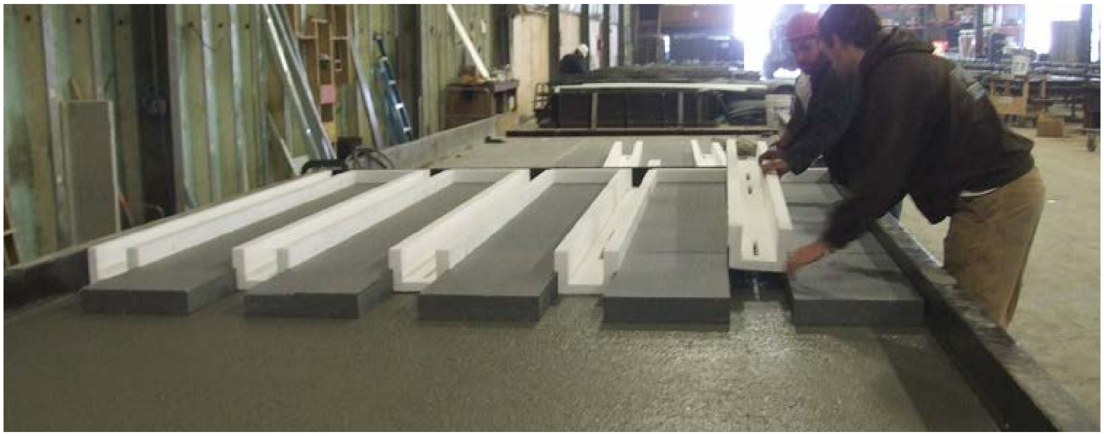
(a)



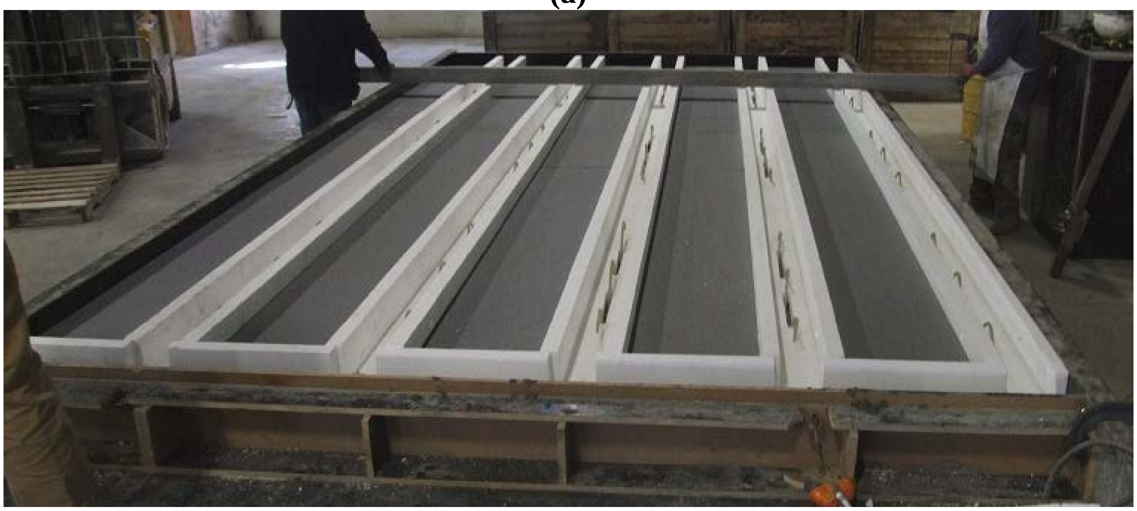
(b)

Figure 5: Truss models used to predict the deflection of the two specimens

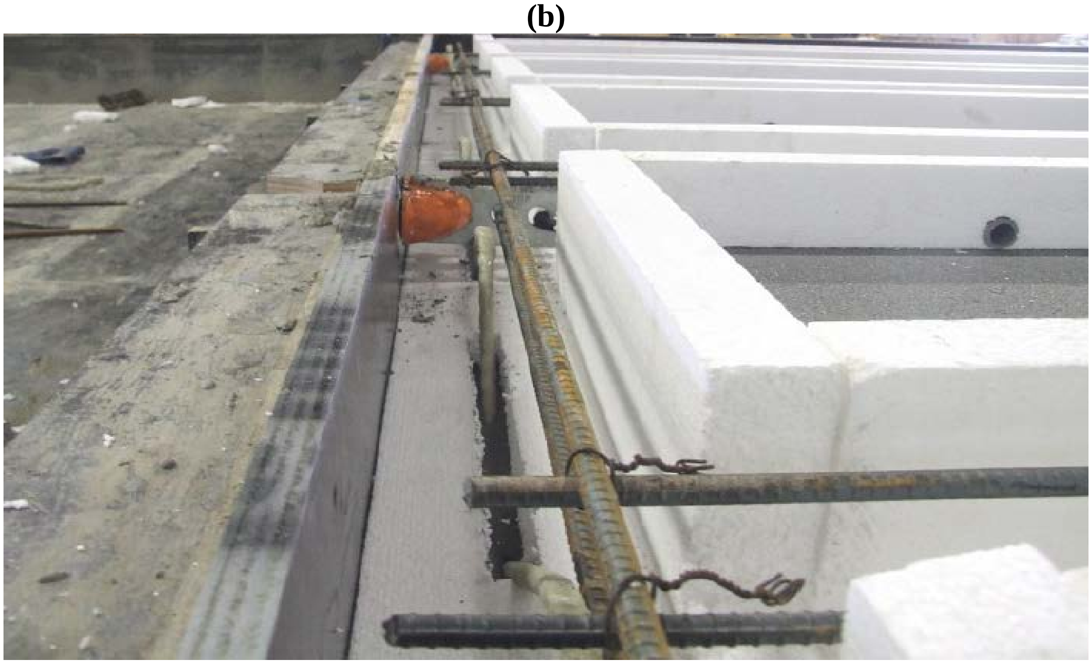
Fabrication of the two specimens using GFRP ties was conducted in a very efficient and unique process. First, the bottom wythe reinforcement is installed. Second, self-consolidated concrete (SCC) with specified 28-day compressive strength of 6,000 psi is cast and leveled. Foam planks and slotted foam tubs were installed as shown in Figure 6(a). Ties were installed, as shown in Figure 6(b) and the gaps in the tie slots were filled with expanded foam to prevent forming concrete connectors and maintain the thermal efficiency of the panel. Then, reinforcement of the studs and lifting inserts were installed as shown in Figure 6(c). Lastly, SCC was cast in all the longitudinal and transversal studs as shown in Figure 6(d) and the completed panel was cured as shown in Figure 6(e).



(a)



(b)



(c)

(d)

(e)

Figure 6: Steps of fabricating the test specimens

TESTING

The cured panels were lifted from the bed, as shown Figure 7, using the lifting inserts embedded in the panel studs and placed on two 6 in. wide wooden supports at the panel ends. The average compressive strength at the time of testing was 5,600 psi. Each panel was tested by three point loading (two end supports and one mid-span loading point) using pre-weighed concrete blocks. The 16 ft long specimen was loaded on the stud side as shown in Figure 8, while the 20 ft long specimen was loaded on the flat side as shown in Figure 9. Testing began by measuring the mid-span panel deflection under its self-weight. Then, the applied load was gradually increased by adding concrete blocks and recording the total load and the corresponding mid-span panel deflection until failure. Test results and failure modes are discussed in the following section.

Figure 7: Handling the 20 ft long test specimen

Figure 8: Loading the 16 ft long on the stud side (16 ft long panel)

Figure 9: Loading the 20 ft long specimen on the flat side

TEST RESULTS

Figure 10 plots the load-deflection relationship at the mid-span of the two panels due to the added loads. Prior to panel loading, the 16 ft and 20 ft long specimens deflected 0.25 in. and 0.375 in. respectively due to their self-weight. Figure 10 indicates that the ultimate load of the 16 ft long panel was 7,700 lb and the corresponding deflection was 2.25 in., while the ultimate load of the 20 ft long panel was 6,025 lb and the corresponding deflection was 3.2 in. These load values correspond to bending moments of approximately 45 kip.ft and 53 kip.ft (including self-weight moment) for the 16 ft and 20 ft panels respectively. These moments exceed the theoretical moment capacity of the two panels by approximately 9% and 4% respectively. Figure 11 shows that the failure mode of the 20 ft long panel was flexural failure due to panel overloading. The panel did not show any signs of horizontal shear failure. On the other hand, Figure 12 shows that the 16 ft long panel has significant relative movement between the bottom concrete wythe and top concrete studs. This movement occurred due to the pullout of the GFRP from the concrete as a result of horizontal shear from excessive loading. Higher embedment of GFRP ties or using more ties would have resulted in additional capacity[6].

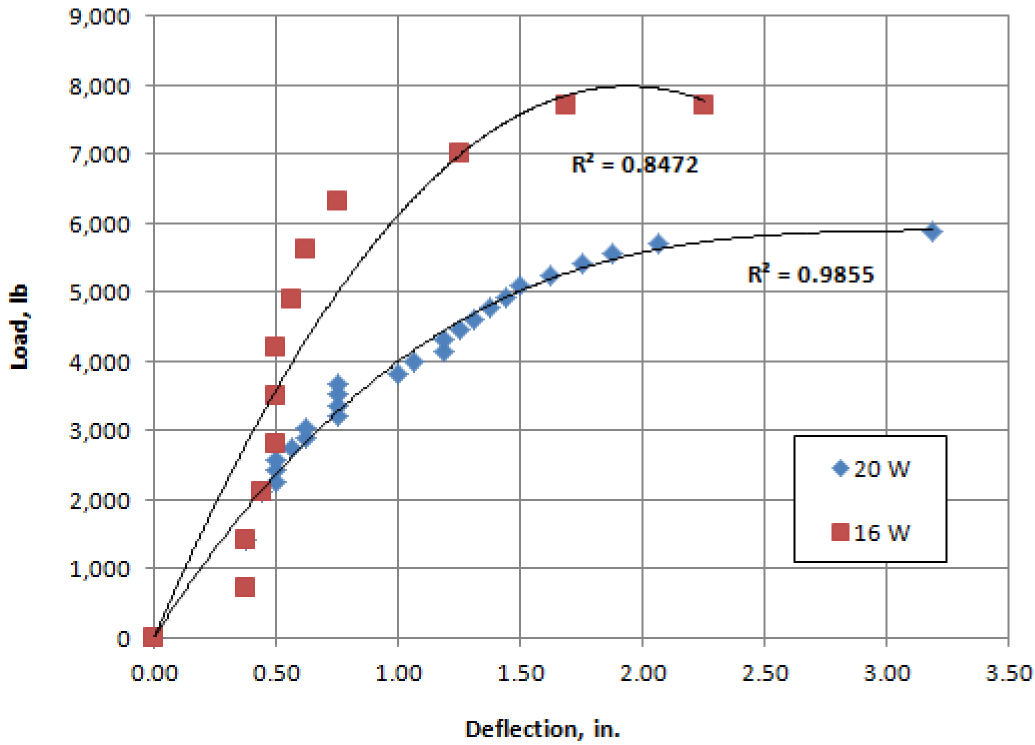


Figure 10: Load-deflection relationship for the two specimens

Figure 11: Flexural failure of the 20 ft long panel due to overloading

Figure 12: Horizontal shear failure of the 16 ft long panel due to the pullout of GFRP ties

Table 1: Theoretical and measured capacity and deflection of tested specimens.

Panel ID	16W	20W
Concrete Strength (psi)	5,600	5,600
Modulus of Rupture (psi)	561	561
Modulus of Elasticity (ksi)	4,265	4,265
Cross Section Area (in.^{2})	480.00	478.75
Self Weight (kip/ft)	0.50	0.50
Inertia (in.^{4})	3,153	3,073
Centroid from Bottom (in.)	2.875	6.417
Span (ft)	15.50	19.50
Cracking Moment (kip.ft)	51.29	22.40
Self-Weight Deflection from Beam Model (in.)	0.05	0.12
Self-Weight Deflection from Truss Model (in.)	0.27	0.48
Measured Self Weight Deflection (in.)	0.25	0.38
Self-Weight Moment (kip.ft)	15.0	23.7
Maximum Applied Load (kip)	7.70	6.03
Moment Due to Maximum Applied Load (kip.ft)	29.84	29.37
Total Moment (kip.ft)	**44.85**	**53.08**
Equivalent Wind Pressure (psf)	**149**	**112**
Nominal Moment Capacity (kip.ft)	**41.20**	**50.80**
Actual / Nominal Moment Capacity (%)	**109%**	**104%**

CONCLUSIONS AND RECOMMENDATIONS

Based on the testing results presented earlier, the following conclusions were made:

1- The flexural design of the tested panels is adequate for resisting the wind pressure of 149 psf for the 16 ft long panel and 112 for the 20 ft long panel, which is significantly higher than the design wind pressure.
2- The number, size, and distribution of GFRP ties are adequate for resisting horizontal shear and achieving the flexural capacity of a fully-composite panel.
3- Horizontal shear failure occurred in one specimen due to the pullout of the ties and not due to overstressed ties, which indicates that a deeper tie embedment is needed to fully utilize the ties.
4- The elastic deflection of the uncracked panel under their self-weight can be accurately predicted using a truss model not the beam model.

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