DESIGN AND TESTING OF TORNADO-RESISTANT PRECAST/PRESTRESSED CONCRETE SANDWICH PANELS WITH GFRP TIES

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Abstract

Glass fiber-reinforced polymer (GFRP) ties are used as structurally and thermally efficient shear connectors for precast concrete sandwich panels (PCSPs) to link the two concrete wythes through a layer of insulation. The excellent mechanical properties and high thermal resistance of these ties have resulted in the growing use of PCSPs in building construction. An emerging application for PCSPs is the wall panels for tornado-resistant buildings (e.g. tornado shelters). The lightweight, structural efficiency, thermal and sound insulation, and speed of construction are the main advantages of these panels. This paper introduces the use of large diameter GFRP ties (5/8 in.) in 14 in. thick (5-4-5) PCSPs to resist the significantly high wind pressure of tornados. The main objective of this research is to experimentally investigate the flexural capacity, horizontal shear capacity, and deflection of tornado-resistant PCSPs using large diameter GFRP ties. A 32 ft long, 4 ft wide, and 14 in. thick panel was fabricated by Concrete Industries Inc., Lincoln, NE using twelve 5/8 in. diameter GFRP ties and tested at the structural laboratory of the University of Nebraska-Lincoln in Omaha. Testing results indicated that the design of the panel is adequate for resisting the tornado wind pressure calculated according to FEMA 361; 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 failure mode of the tested panel was the pullout of ties rather than the rupture of ties, which indicated that deeper embedment of ties is needed to fully utilize the GFRP ties.

Keywords: Concrete Sandwich Panel, GFRP Ties, Thermal Efficient, Composite, Tornado

INTRODUCTION

Precast concrete sandwich panel (PCSP) is 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 [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 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. A new system was developed and patented by researchers at the University of Nebraska-Lincoln (UNL) in August 15, 1995.[2] In this new system, connectors are made of glass fiber-reinforced polymer (GFRP) bars due to their excellent thermal and mechanical properties.[3]

During the last decade, several research experiments were conducted to investigate the structural performance of PCSP panels using different designs of GFRP connectors. This 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). Figure 1 shows the different generations of GFRP ties. The investigation presented in this paper uses the fifth generation of GFRP ties.

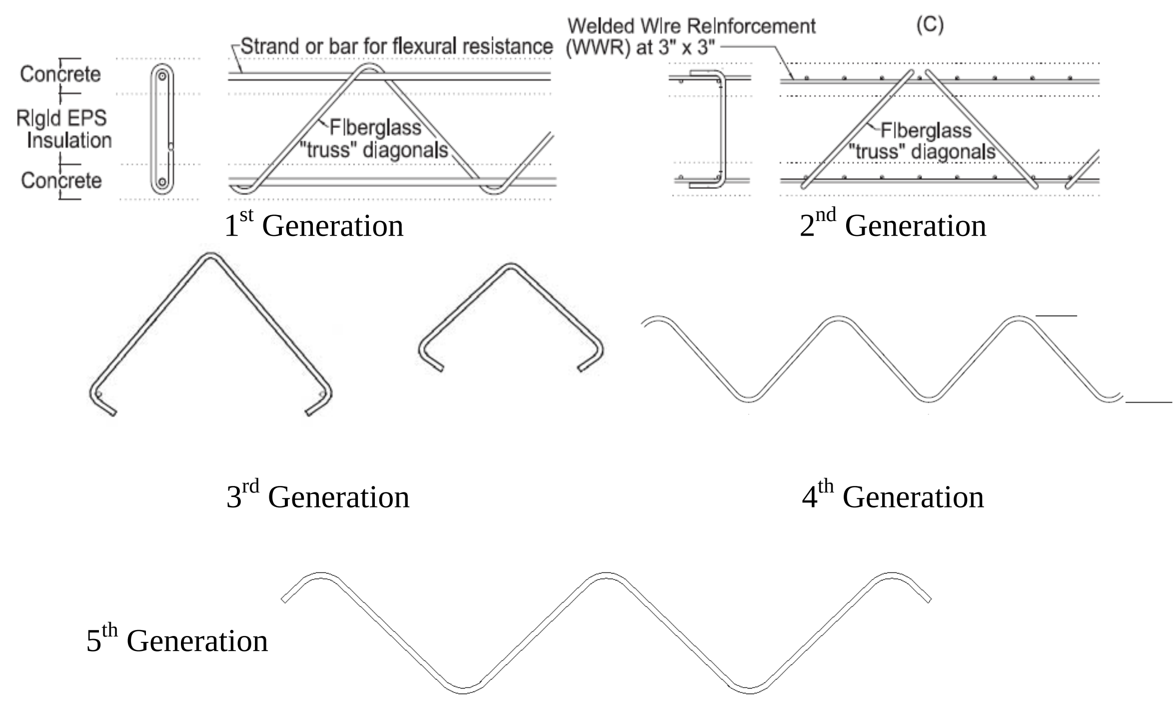


Fig. 1: Generations of GFRP connectors

Several experiments were conducted on the fifth generation of GFRP ties to investigate the optimal distribution of ties, level of composite action achieved, and minimum required embedment depth of ties.[4] An emerging application for PCSP with GFRP ties is the wall panels of tornado-resistant buildings, such as tornado shelters. The lightweight, structural efficiency, thermal and sound insulation, and speed of construction are the main advantages of these panels. This application requires the use of large diameter GFRP ties and thick concrete wythes to resist the significantly wind pressure of 211 psf. Lateral wind tests of this magnitude have not been tested before. The GFRP ties used in this investigation are made of 5/8-in. diameter GFRP bent rods, which have a cross-sectional area of 0.31 in^2 and profile as shown in Figure 2. All earlier investigations were conducted using 3/8-in. GFRP tie bars. 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 a guaranteed tensile strength of 95 ksi, ultimate tensile load of 29.1 kip, and average modulus of elasticity of 5,920 ksi for 5/8 in. diameter ties.

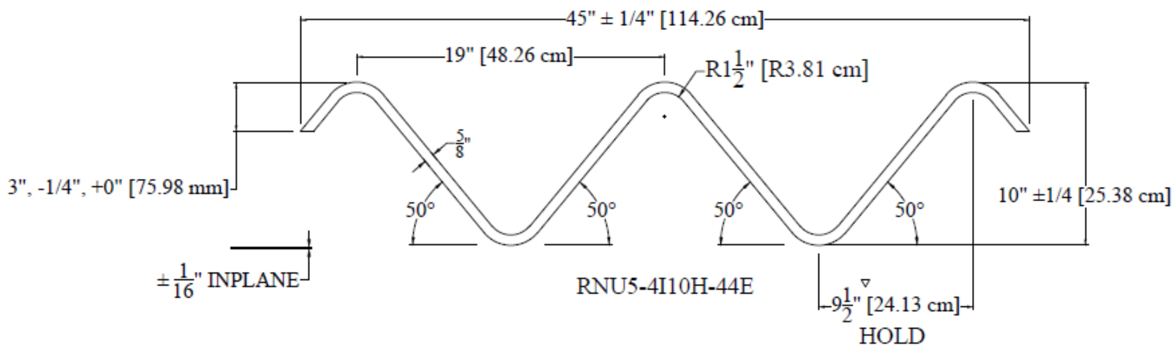


Figure 2: Profile and dimensions of 5/8 in. diameter GFRP tie

The objective of this research is to experimentally investigate the flexural and shear capacity and behavior of tornado-resistant PCSPs made of 5/8 in. diameter GFRP-ties. The next section presents the analysis and design of the test specimen, while the section following presents the fabrication and testing of the specimen. The last section summarizes test results and presents research conclusions.

ANALYSIS AND DESIGN

A 32 ft long, 4 ft wide, and 14 in. thick (5-4-5) sandwich panel was designed as a test specimen for evaluating the flexural and horizontal shear capacities. This panel was required to resist the tornado load combination (0.9D+1.2W) calculated according to the Federal Emergency Management Agency (FEMA) Design and Construction Guidance for Community Shelters FEMA 361, July, 2000. This load combination results in an ultimate moment (M_u) of 115.2 kip.ft considering a tornado wind pressure of 211 psf and P-Delta effects of the wall panel.[6] The panel was designed to resist this load effect using 8 ksi concrete and 3-0.5 in. Grade 270 prestressing strands in each wythe initially tensioned to

$0.7f_{pu}$. The two wythes were also transversally reinforced using #3 deformed bars @32 in. and connected using twelve 5/8 in. diameter GFRP ties distributed as shown in Figure 3. The nominal flexural capacity of this panel (M_n) was estimated at 136.9 kip.ft assuming a full-composite action.

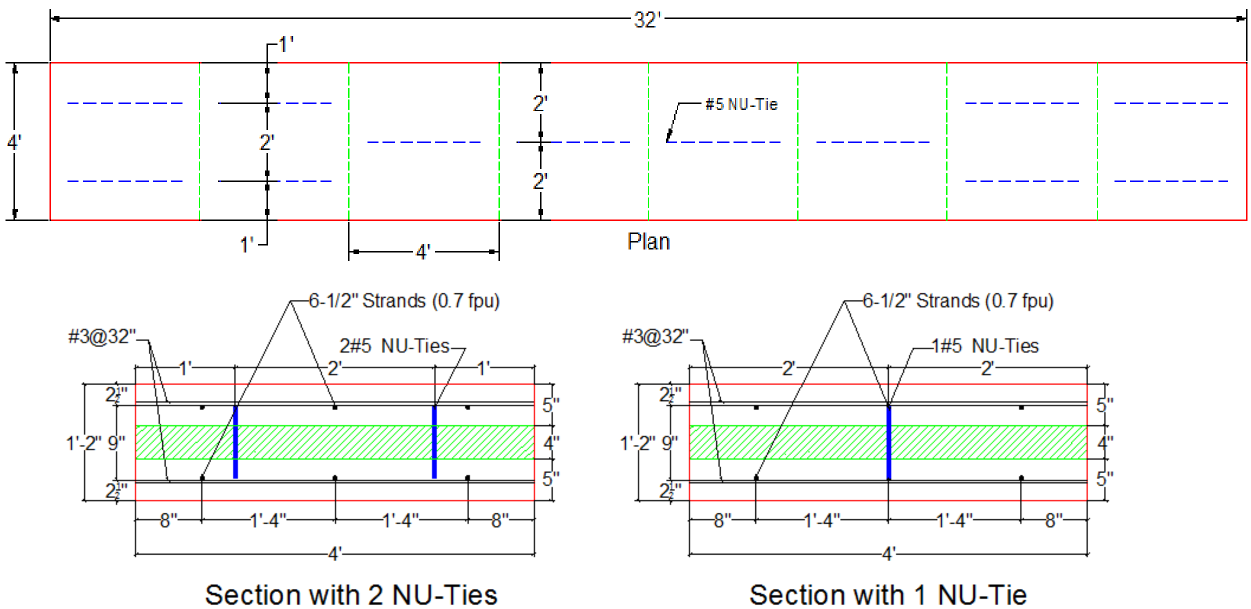


Figure 3: Plan and Sections of the test specimen

A truss model was developed to predict the panel deflections under its self-weight and cracking loads. In this model, shown in Figure 4, the top and bottom concrete wythes were modeled as the truss top and bottom chord members, while the diagonal members modeled the GFRP ties. Figures 5(a), and (b) shows that the mid-span panel deflections were 0.575 in. and 0.5 in. under self-weight and cracking load respectively. These predicted deflections will be compared versus the measured deflections of the test specimen in the next section.

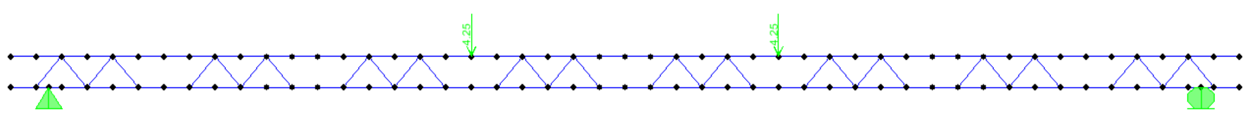
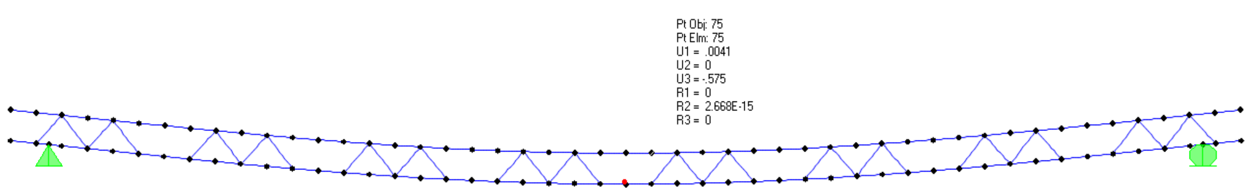
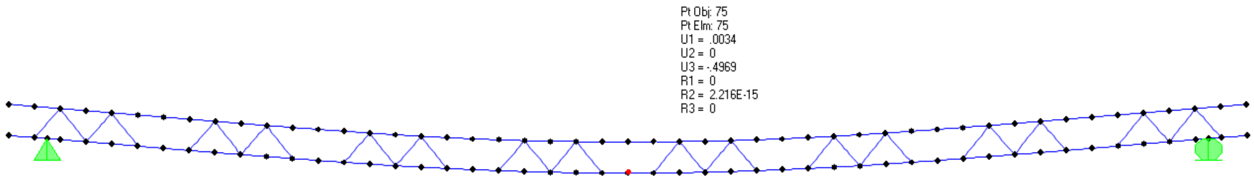


Figure 4: Truss model used to predict panel deflection



(a)

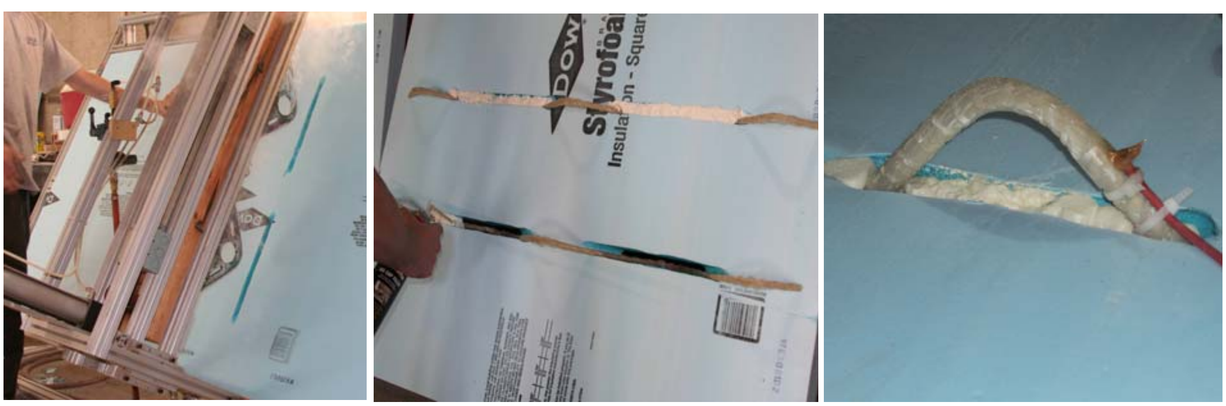


(b)

Figure 5: Panel deflection under a) self-weight; and b) cracking load

FABRICATION AND TESTING

Fabrication of the PCSP using GFRP ties is conducted in a very efficient and unique process. First, the bottom wythe reinforcement is installed and strands are tensioned. Second, self-consolidated concrete (SCC) is cast and leveled. Foam panels are slotted using special melting equipment as shown in Figure 6a. Ties are installed and the gaps in the tie slots are filled with expanded foam, as shown in Figure 6b, to prevent forming concrete connectors and maintain the thermal efficiency of the panel. Last, reinforcement and prestressing of the top wythe is installed and SCC is cast and leveled similar to the bottom wythe. For more details on this process, refer to Morcous, et al. 2010[7]. Strain gauges were connected to the tension legs of the GFRP ties before concrete casting as shown in Figure 6c. These gauges were distributed and labeled as shown in Figure 7. The specified 28-day compressive strength of the panel concrete was 8,000 psi and strands were released after one-day when concrete strength was 5,500 psi. Two concrete strain gauges were attached to the top surface of the hardened panel and one potentiometer was attached to the bottom surface at mid-span as shown in Figure 7.



a) b) c)

Figure 6: Steps of fabricating the test specimen

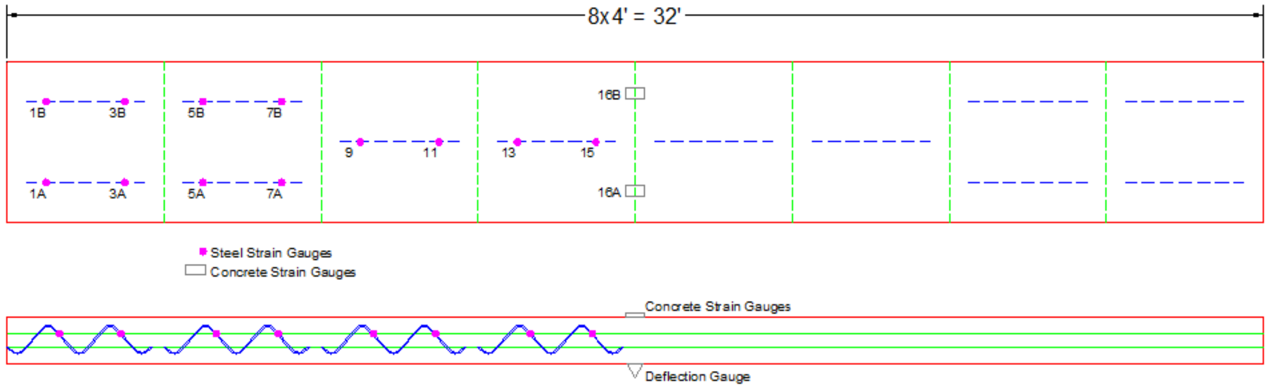


Figure 7: Specimen instrumentation

Figure 8 shows the test setup of the specimen, which is a four point loading of a 30 ft long span. Since the self-weight of the panel is significant (125 psf), a temporary support was placed under the panel at the mid-span, as shown in Figure 9, to eliminate the panel deflection under self-weight and prevent any creep. Testing began by removing the temporary support while recording the deflection and strains on the GFRP ties and concrete surface due to self-weight. Then, the applied load was gradually increased until the loading jack ran out of stroke. The panel was unloaded and steel plates were added to lower the loading jack. The panel was reloaded as shown in Figure 10, up to failure, which occurred due to horizontal shear that caused the pullout of some ties from the bottom concrete wythe as shown in Figure 10. No measurement of the relative movement of the two concrete wythes were taken.

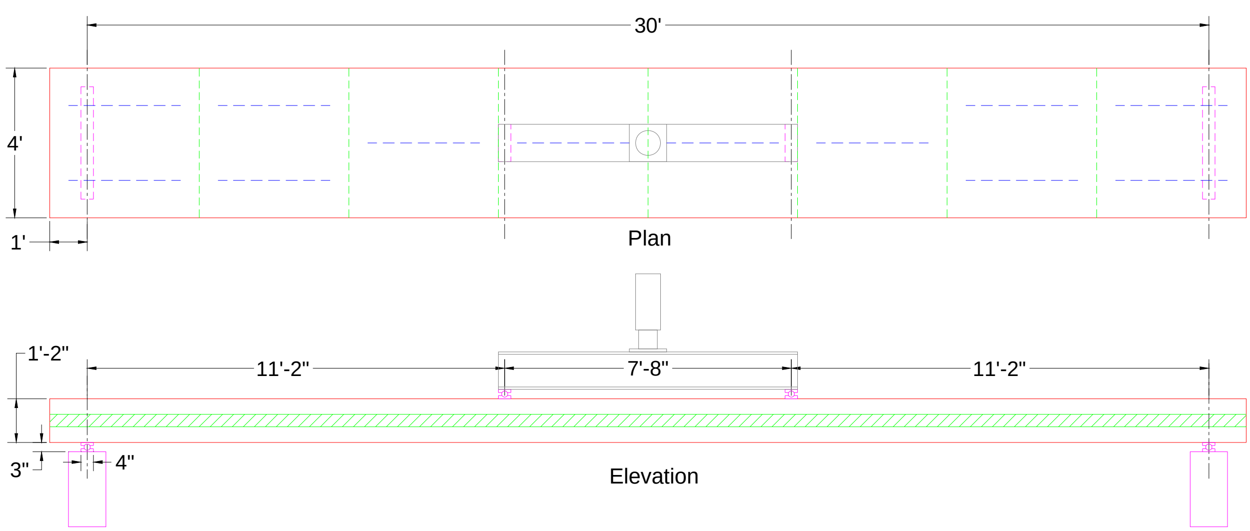


Figure 8: Test setup

Figure 9: Test specimen with temporary support at mid-span

Figure 10: Pull out of GFRP tie at failure

TEST RESULTS

Figure 11 shows the compressive strength of the concrete used in the test panel versus age up to the time of testing (49 days). The average compressive strength of three cylinders was found to be approximately 11,000 psi, which is more than the specified 28-day strength of 8,000 psi.. Figure 12 plots the load-deflection relationship at the mid-span of the panel. This figure indicates that the panel deflected approximately 0.4 in. under its self-weight and approximately 1.5 in. under the ultimate design load (10.0 kip). Panel deflection continued to increase up to 10 in. as the load reached 15.6 kips, which corresponds to a bending moment of 148.7 kip.ft. This moment exceeds the theoretical moment capacity of the panel by approximately 8%. When the panel was unloaded, a 6 in. permanent deflection was observed, while a 4 in. deflection was recovered. Then, the panel was re-loaded up to failure, which occurred at a load of 16 kips and 10.2 in. deflection as shown by the red curve in Figure 12.

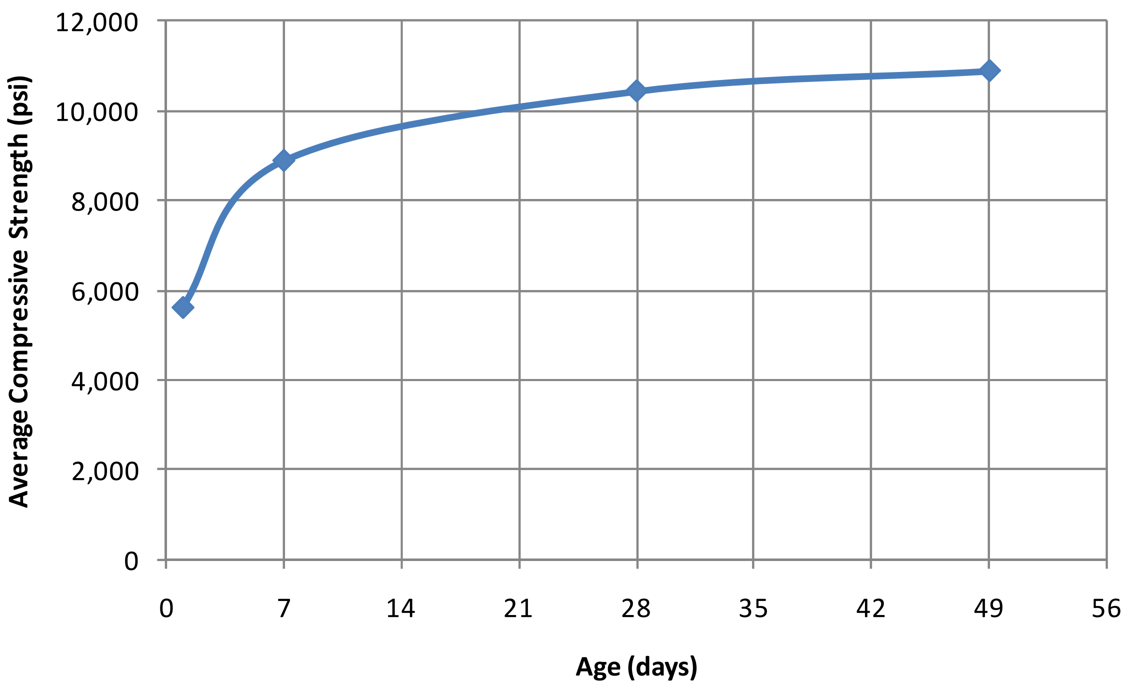


Figure 11: Compressive strength vs. age of panel concrete

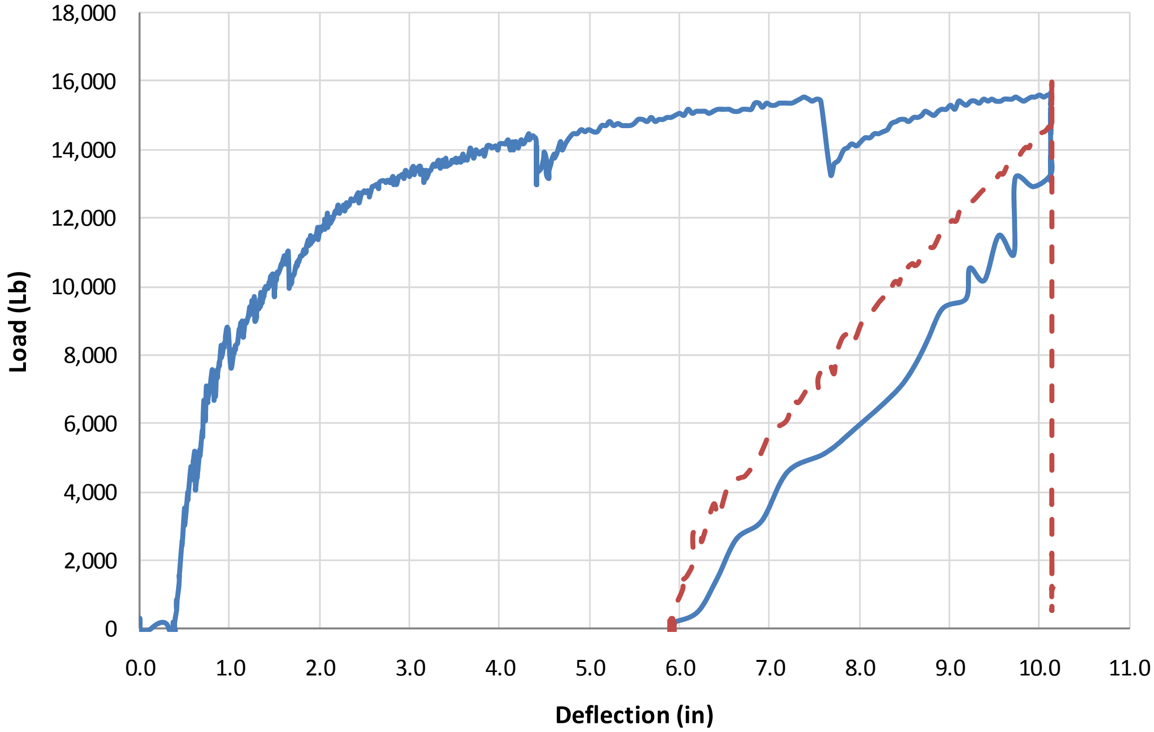


Figure 12: Load-deflection relationship for the first (solid) and second (dashed) loadings

Figures 13 and 14 plot the load-strain relationships at the top concrete surface and in the tension legs of several GFRP ties respectively. Figure 13 indicates that the maximum compressive strain in the concrete at mid-span was 0.0018, It also shows a sudden increase in the strain at a load of 8.7 kip, which is considered to be the cracking load (corresponds to a cracking moment of 110 kip.ft). Figure 14 indicates that the maximum strain in the GFRP ties is approximately 0.0063, which occurred at the ties located 3 ft from the panel end. This strain corresponds to a stress of approximately 37.3 ksi using modulus of elasticity of 5,920 ksi. This stress level is below the design stress of the ties after considering the exposure and interaction coefficients (95 x 0.7 x 0.65 = 43 ksi). All the other ties have strains less than 0.0063 even under ultimate loads, which indicates that the 5/8 in. diameter GFRP ties used are not fully utilized. Figure 14 also indicates that the ties closer to the end of the panel have higher strains than those closer to the middle of the panel, except for those located at 9 ft from the panel end where the number of ties changes from two to one. This change results in a sudden increase in the strain values. It also should be noted that the horizontal shear distribution in the tested panel is the combination of the triangular distribution due to self-weight and the rectangular distribution due to applied load, which explains why the strain values are not linearly proportioned to the tie location. The measured mid-span deflections under the self-weight and cracking load were found to be 0.4 in. and 0.5 in., respectively. Comparing the predicted and measured cracking load deflections indicates that the truss model accurately predicts the behavior of uncracked panel. Note the difference between the predicted and measured self-weight deflections might be due to the inaccuracy of leveling the panel before measuring it deflection.

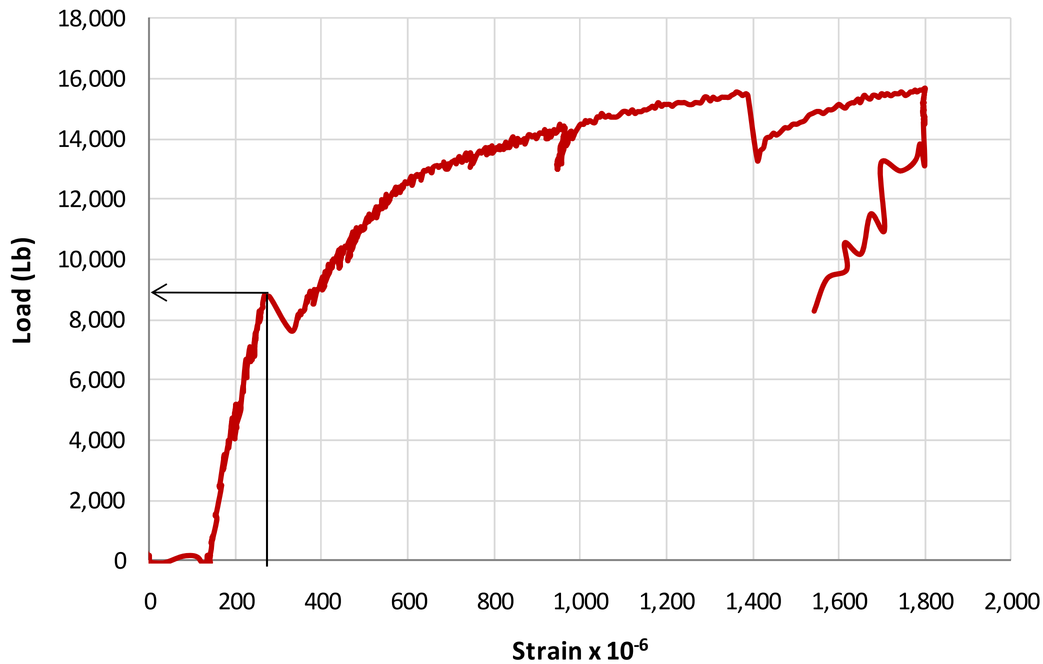


Figure 13: Load-strain relationship at the top concrete surface

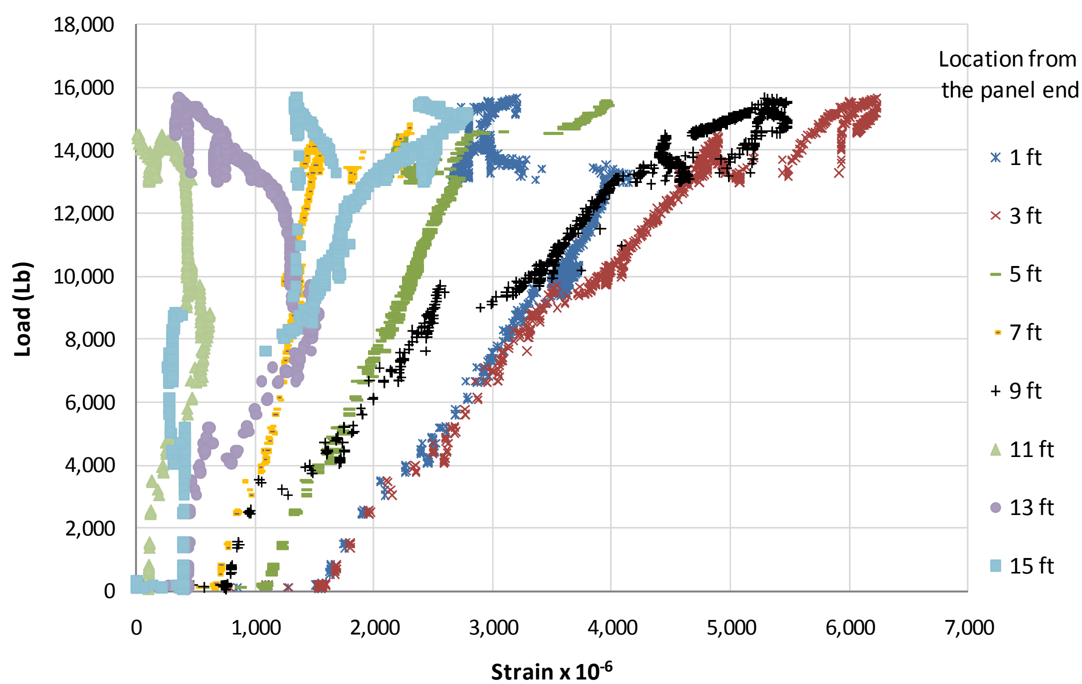


Figure 14: Load-strain relationship for GFRP ties at different locations

CONCLUSIONS AND RECOMMENDATIONS

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

1- The flexural design of the tested tornado-resistant sandwich panel is adequate for resisting the tornado wind pressure calculated based on FEMA 361 (2000) " Design and Construction Guidance for Community Shelters." Table 1 compares the required, designed, and tested flexural capacities.
2- The number, size, and distribution of GFRP ties used in the tested tornado-resistant sandwich panel are adequate for resisting horizontal shear and achieving the flexural capacity of a fully-composite panel.
3- The horizontal shear failure occurred due to the pull out of the ties from their embedment in the concrete and not due to overstressed ties, which indicates that a deeper tie embedment is needed.
4- The elastic deflection of the uncracked panel can be accurately predicted using a truss model, where top and bottom concrete wythes are modeled as truss top and bottom chord members, while GFRP ties are modeled as diagonal members

Table 1: Comparing calculated and measured flexural capacity

Flexural Capacity (kip.ft)				
Calculated			Measured	
Demand	Cracking	Nominal	Cracking	Ultimate
115.2	103	136.9	110	148.7

Based on the above conclusions, tornado-resistant sandwich wall panels can be designed using 5/8 in. diameter GFRP ties. According to the results of testing the minimum embedment depth for 2/8, 3/8, and 4/8 in. diameter GFRP ties conducted by Morcous and Henin[4] in an earlier study, the minimum embedment depth for 5/8 in. diameter GFRP ties can be predicted by extrapolation to be 4 inches. This recommendation is currently being confirmed through an additional experimental investigation.

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